



**Washington State  
Department of Transportation**

**SEG 2B  
RETAINING WALL 09.90L - A & B**

**I-405; RENTON TO BELLEVUE WIDENING AND  
EXPRESS TOLL LANES PROJECT**

**Design Calculations**

***FINAL SUBMITTAL***



December 2021



600 University Street, Ste 700  
Seattle, Washington 98101



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### RETAINING WALL 09.90L – A & B

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## **RETAINING WALL 09.90L - A & B**

*1.0 — Lightweight Fill Design*



## **RETAINING WALL 09.90L - A & B**

### *1.1 — External Stability Evaluation*

<b>PARSONS</b>		MADE BY	DATE	CHK BY	DATE
Job Number	WBS Number	W. Metwally	11/12/2021	E. Kelley	11/21/2021
650512	00531	TITLE	I-405 Renton to Bellevue		
			Lightweight Fill - External Stability Evaluation		

**[A] BASIS**

Design for the external global stability of the EPS-Geofoam vertical embankment involves consideration of how the combined embankment system interacts with the proposed foundation soil. External stability of the vertical embankment consider issues at the serviceability state and ultimate limit state.

**[B] REFERENCES**

Acronym	Source
AASHTO	AASHTO LRFD Bridge Design Specifications (8th Edition)
BDM	WSDOT Bridge Design Manual M23-50.18 (2018 Edition)
NCHRP Project No. 24-11(020)	Guidelines for Geofoam Applications in Slope Stability Projects
GD	Wall Package 1 - Retaining walls 9.90L-A and 9.90L-B Geotechnical Design Memorandum

**[C] CALCULATION LEGEND**

-Standard cell format for User Input, Check Passed, No Good, and Output, respectively:

User Input	OK	NG	Output
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-Curly brackets, { }, are used to enclose descriptions of inputs and calculations

-Square brackets, [ ], are used to enclose references

-All stations (#.# #####.###) have units 'ft.' Other units are noted with their respective values.

**[D] MATERIAL PROPERTIES**

$\gamma_{c,E}$ =	150 lb/ft <sup>3</sup>	{reinforced concrete unit weight for capacity analysis}	[BDM Table 3.8-1]
$\gamma_c$ =	145 lb/ft <sup>3</sup>		[AASHTO-Table 3.5.1-1]
$\gamma_{pav}$ =	140 lb/ft <sup>3</sup>	{pavement unit weight}	[AASHTO-Table 3.5.1-1]
$\gamma_{soil}$ =	120 lb/ft <sup>3</sup>	{soil unit weight}	
$\gamma_{EPS}$ =	1.80 lb/ft <sup>3</sup>	{EPS unit weight}	[ASTM D6817]

**Unconfined Concrete**

$f'_c$	$K_1$	$E_c$	$f_t$	$\epsilon_{cu}$	$v_c$	$G_c$
4.0 ksi	1.00	4266.2 ksi	0.48 ksi	0.003	0.2	1777.6 ksi

$$f'_c = \{ \text{concrete compressive strength} \}$$

$$E_c = 120,000 K_1 \gamma_c^2 f'_c^{0.33}$$

$$f_t = 0.24 \sqrt{f'_c} \quad \{ \text{concrete tension capacity for this analysis} \}$$

$$\epsilon_{cu} = \{ \text{maximum unconfined concrete strain} \}$$

$$v_c = \{ \text{poisson's ratio for concrete} \}$$

$$G_c = E_c / (2 * (1 + v_c)) \quad \{ \text{concrete shear modulus} \}$$

[AASHTO-5.4.2.4-1]

[AASHTO-5.4.2.6]

[AASHTO-5.6.2.1]

[AASHTO 5.4.2.5]

**ASTM A706 Grade 60 Reinforcing Steel**

$f_y$	$f_u$	$E_s$	$\epsilon_y$
60.0 ksi	80.0 ksi	29000 ksi	0.0021

$$f_y = \{ \text{minimum yield strength} \}$$

$$f_u = \{ \text{minimum tensile strength} \}$$

$$E_s = \{ \text{steel modulus of elasticity} \}$$

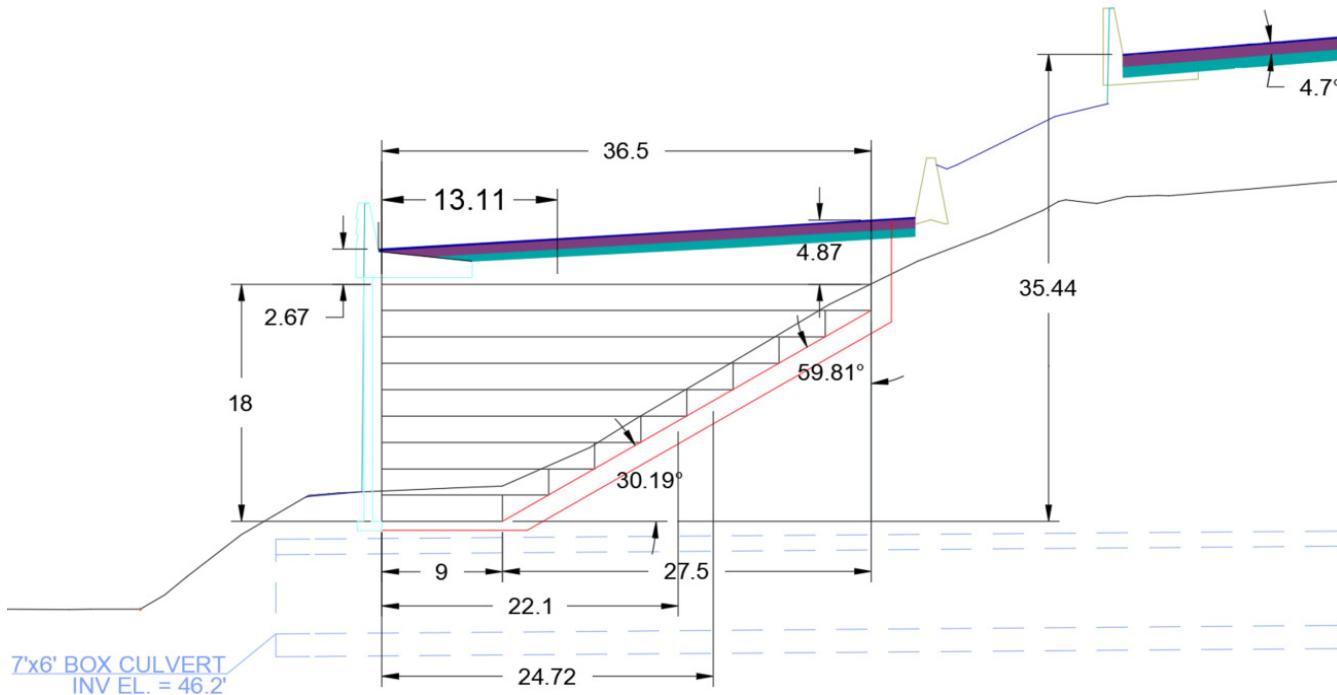
$$\epsilon_y = \{ \text{nominal yield strain} \}$$

[ASTM A706-16 Table A1.2]

[ASTM A706-16 Table A1.2]

[AASHTO 5.4.3.2]

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		TITLE	I-405 Renton to Bellevue Lightweight Fill - External Stability Evaluation		
650512	00531				

[E] Vertical Embankment Geometric Properties

H <sub>EPS</sub> =	18.00 ft	{EPS height}
H <sub>fill(MIN)</sub>	2.17 ft	{Minimum fill height above the EPS}
H <sub>fill(MAX)</sub>	4.37 ft	{Maximum fill height above the EPS}
H <sub>fill (behind wall)</sub>	35.44 ft	{Maximum fill height behind the EPS}
W <sub>top</sub> =	36.50 ft	{EPS width at top}
W <sub>bot</sub> =	9.00 ft	{EPS width at bottom}
L <sub>EPS</sub> =	30.00 ft	{EPS length}
H <sub>pav. Avg.</sub> =	3.27 ft	{Average fill height above the EPS}
H <sub>active soil</sub>	22.37 ft	{Active soil behind the wall}
H <sub>ES</sub> =	13.07 ft	{Uniform Surcharge Load height}
h <sub>LL</sub> =	2.00 ft	{Live load surcharge height}
EPS overturning arm=	13.10 ft	{Using Autocad}
T <sub>slab</sub> =	0.50 ft	{Distribution Slab thickness}

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650512	00531				

[F] Sliding Due to the Backfill Material

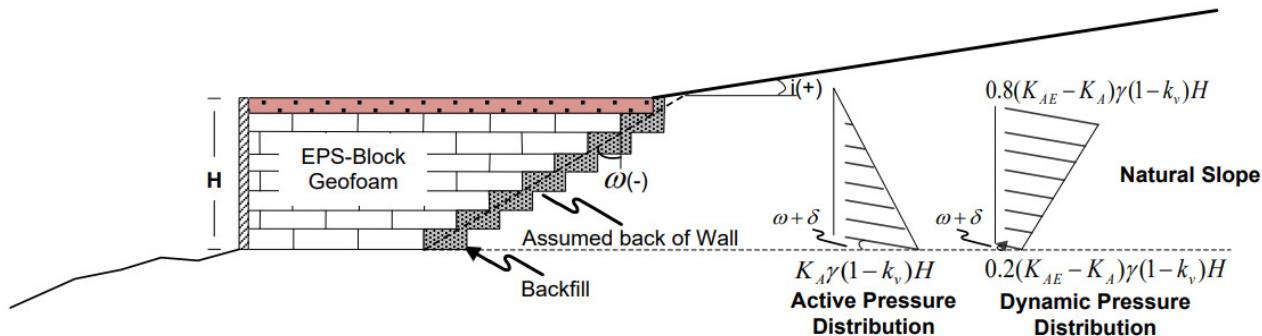


Figure B.15. Static and dynamic components of active earth pressure.

$A_{EPS} =$	409.50 ft <sup>2</sup>	{Area of EPS fill}
$W_{EPS} =$	737.10 lb	{weight of EPS fill per linear foot of fill}
$Aslab =$	18.25 ft <sup>2</sup>	
$Wslab =$	2737.50 lb	
$A_{Pavement} =$	119.36 ft <sup>2</sup>	
$W_{Pavement} =$	14322.60 lb	
$W_{pavement+Slab} =$	17060.10 lb	
$\delta =$	22.00	
$\mu =$	0.40	
$\phi =$	32.00	
$\omega =$	-59.81	
$i =$	4.70	
$\delta =$	16.00	
$K_A =$	0.0013	NCHRP Eq. 4.23
$P_A =$	39.15 lb	NCHRP Figure 4.29]
$\gamma_{p,EH} =$	1.50	[AASHTO Table 3.4.1-1]
$P_{AX} =$	42.38 lb	
$\gamma_{p,ES} =$	1.50	[AASHTO Table 3.4.1-1]
$P_{ES} =$	68.62 lb	
$\gamma_{p,LS} =$	1.75	[AASHTO Table 3.4.1-1]
$P_{LS} =$	12.25 lb	
$\Sigma P =$	123.25 lb	
$\phi_r =$	0.90	
$\gamma_{p,EV} =$	1.00	AASHTO Table 3.4.1-1]
$\gamma_{p,DC} =$	0.90	
Sliding FS=	51.700	OK {Factor of safty >1.2}

[G] Overturning Due to the Backfill Material

$M_o =$	1220.53 lb.ft	{Overturning moment}
$M_R =$	315409.20 lb.ft	{Resist moments}
Overturning FS=	258.420	OK {Factor of safty >1.2}

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[H] Seismic Horizontal Sliding

PGA=	0.500	g	{Peak ground acceleration}	[GD Table 4]
Kh=	0.250		{horizontal seismic coefficient}	
$\Psi$ =	14.036		{Angle of the resultant body force in the soil from vertical}	
K <sub>AE</sub> =	0.0750		{Earthquake active earth pressure coefficient}	[NCHRB Eq. 4.21]
P <sub>AE</sub> =	2213.10 lb		{seismic force}	[NCHRB Figure 4.29]
$\Sigma P$ =	2252.24 lb		{P <sub>A</sub> +P <sub>AE</sub> }	
$\gamma_{p,EH}$ =	1.00		{Earth horizontal load factor}	[AASHTO Table 3.4.1-1]
$\gamma_{p,ES}$ =	1.00		{Uniform surcharge load factor}	[AASHTO Table 3.4.1-1]
$\gamma_{LS}$ =	0.50		{Live load surcharge load factor}	[AASHTO Table 3.4.1-1]
$\varnothing_r$ =	1.00		{Resistance factor}	
$\gamma_{p,EV}$ =	1.00		{Earth vertical load factor}	[AASHTO Table 3.4.1-1]
$\gamma_{p,DC}$ =	1.00		{Dead load load factor}	[AASHTO Table 3.4.1-1]
$\gamma_{EQ}$ =	1.00		{Earthquake load factor}	[AASHTO Table 3.4.1-1]
$\Sigma P_x$ =	1674.56 lb		{Horizontal component of $\Sigma P$ }	
$\Sigma P_y$ =	1559.16 lb		{Vertical component of $\Sigma P$ }	
Sliding FS=	1.277	OK	{Factor of safety >1.2}	

[I] Seismic Overturning

M <sub>o</sub> =	15051.96 lb.ft	{Overturning moment}
M <sub>R</sub> =	283543.83 lb.ft	{Resist moments}
Overturning FS=	18.838	OK {Factor of safety >1.2}

[J] Horizontal Sliding due to collision force

L <sub>s</sub> =	10000.00 lb	{Collision equivalent static load}	[BDM 10.3.2-2]
L <sub>slab,min</sub> =	40.00 ft	{minimum slab length}	
$\gamma_{p,EH}$ =	1.00	{Earth horizontal load factor}	[AASHTO Table 3.4.1-1]
$\gamma_{p,ES}$ =	1.00	{Uniform surcharge load factor}	[AASHTO Table 3.4.1-1]
$\gamma_{p,LS}$ =	0.50	{Live load surcharge load factor}	[AASHTO Table 3.4.1-1]
$\gamma_{CT}$ =	1.00	{Collision Force load factor}	[AASHTO Table 3.4.1-1]
$\Sigma P$ =	327.50 lb	{ $\Sigma$ Horizontal component of PA}	
$\varnothing_r$ =	1.00	{Resistance factor}	
$\gamma_{p,EV}$ =	1.00	{Earth vertical load factor}	[AASHTO Table 3.4.1-1]
$\gamma_{p,DC}$ =	1.00	{Dead load load factor}	[AASHTO Table 3.4.1-1]
Sliding FS=	21.956	OK {Factor of safety >1.2}	

[K] Overturning due to collision force

$h_{barrier}$ =	3.50 ft	{barrier height total}
$h_a$ =	23.67 ft	{moment arm = top of barrier to point of rotation}
M <sub>o</sub> =	6678.99 lb.ft	{Overturning moment}
M <sub>R</sub> =	320405.14 lb.ft	{Resist moments}
Overturning FS=	47.972	OK {Factor of safety >1.2}

[L] External Seismic Bearing Capacity

Refer to the geotechnical report

[M] Seismic Settlement

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			Lightweight Fill - External Stability Evaluation		

Refer to the geotechnical report

[N] Seismic Settlement

Refer to the geotechnical report



## **RETAINING WALL 09.90L - A & B**

### **1.2 — *Internal Stability Evaluation***

<b>PARSONS</b>		MADE BY	DATE	CHK BY	DATE		
W. Metwally	11/12/2021	E. Kelley	11/21/2021				
Job Number	WBS Number	TITLE	I-405 Renton to Bellevue				
650512	00531		Lightweight Fill - Internal Stability Evaluation				

**[A] BASIS**

Design for the internal global stability of the EPS-Geofoam vertical embankment involves consideration of how the EPS block geofoam behaves under certain loading conditions. Internal stability in the proposed design procedure includes consideration at the serviceability and ultimate limit states

**[B] REFERENCES**

Acronym	Source
AASHTO	AASHTO LRFD Bridge Design Specifications (8th Edition)
BDM	WSDOT Bridge Design Manual M23-50.18 (2018 Edition)
NCHRP Project No. 24-11(020)	Guidelines for Geofoam Applications in Slope Stability Projects
GD	Wall Package 1 - Retaining walls 9.90L-A and 9.90L-B Geotechnical Design Memorandum

**[C] CALCULATION LEGEND**

-Standard cell format for User Input, Check Passed, No Good, and Output, respectively:

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-All stations (##+##.###) have units 'ft.' Other units are noted with their respective values.

**[D] MATERIAL PROPERTIES**

$\gamma_{c,E} =$	150 lb/ft <sup>3</sup>	{reinforced concrete unit weight for capacity analysis}	[BDM Table 3.8-1]
$\gamma_c =$	145 lb/ft <sup>3</sup>		[AASHTO-Table 3.5.1-1]
$\gamma_{pav} =$	140 lb/ft <sup>3</sup>	{pavement unit weight}	[AASHTO-Table 3.5.1-1]
$\gamma_{soil} =$	120 lb/ft <sup>3</sup>	{soil unit weight}	
$\gamma_{EPS} =$	1.80 lb/ft <sup>3</sup>	{EPS unit weight}	[ASTM D6817]

**Unconfined Concrete**

$f_c$	$K_1$	$E_c$	$f_r$	$\epsilon_{cu}$	$v_c$	$G_c$
4.0 ksi	1.00	4266.2 ksi	0.48 ksi	0.003	0.2	1777.6 ksi

$$f_c = \{\text{concrete compressive strength}\}$$

$$E_c = 120,000 K_1 \gamma_c^2 f_c^{0.33}$$

$$f_r = 0.24 \sqrt{(f_c)} \quad \{\text{concrete tension capacity for this analysis}\}$$

$$\epsilon_{cu} = \{\text{maximum unconfined concrete strain}\}$$

$$v_c = \{\text{poisson's ratio for concrete}\}$$

$$G_c = E_c / (2 * (1 + v_c)) \quad \{\text{concrete shear modulus}\}$$

[AASHTO-5.4.2.4-1]

[AASHTO-5.4.2.6]

[AASHTO-5.6.2.1]

[AASHTO 5.4.2.5]

**ASTM A706 Grade 60 Reinforcing Steel**

$f_y$	$f_u$	$E_s$	$\epsilon_y$
60.0 ksi	80.0 ksi	29000 ksi	0.0021

$$f_y = \{\text{minimum yield strength}\}$$

[ASTM A706-16 Table A1.2]

$$f_u = \{\text{minimum tensile strength}\}$$

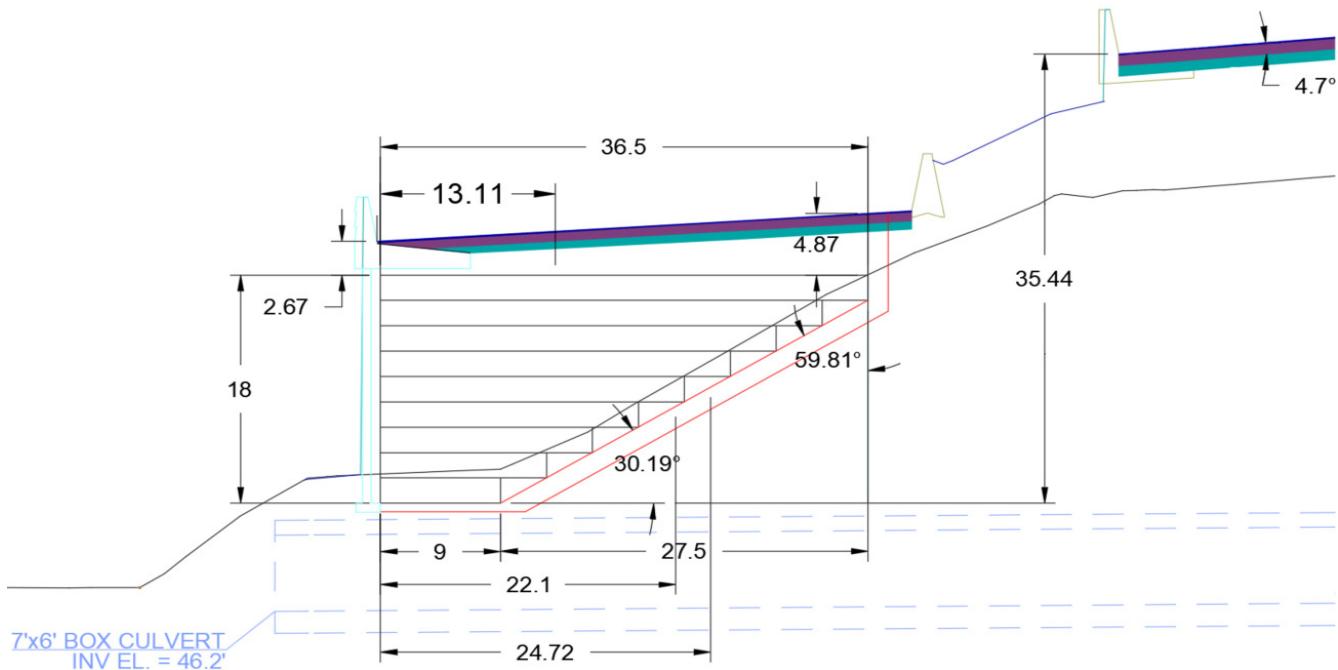
[ASTM A706-16 Table A1.2]

$$E_s = \{\text{steel modulus of elasticity}\}$$

[AASHTO 5.4.3.2]

$$\epsilon_y = \{\text{nominal yield strain}\}$$

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650512	00531		Lightweight Fill - Internal Stability Evaluation		

[E] Vertical Embankment Geometric Properties

$H_{EPS} =$	18.00 ft	{EPS height}
$H_{fill(MIN)}$	2.17 ft	{Minimum fill height above the EPS}
$H_{fill(MAX)}$	4.37 ft	{Maximum fill height above the EPS}
$H_{fill}$ (behind wall)	35.44 ft	{Maximum fill height behind the EPS}
$W_{top} =$	36.50 ft	{EPS width at top}
$W_{bot} =$	9.00 ft	{EPS width at bottom}
$L_{EPS} =$	30.00 ft	{EPS length}
$H_{pav. Avg.} =$	3.27 ft	{Average fill height above the EPS}
$H_{active soil}$	22.37 ft	{Active soil behind the wall}
$H_{ES} =$	13.07 ft	{Uniform Surcharge Load height}
$h_{eq} =$	2.00 ft	{Live load surcharge height}
$EPS$ overturning arm =	13.10 ft	
$T_{slab} =$	0.50 ft	{Distribution Slab thickness}

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			Lightweight Fill - Internal Stability Evaluation		

## [F] Seismic Horizontal Sliding

A <sub>EPS</sub> =	409.50 ft <sup>2</sup>	{Area of EPS fill}	
W <sub>EPS</sub> =	737.10 lb	{weight of EPS fill per linear foot of fill}	
Aslab=	18.25 ft <sup>2</sup>		
W <sub>slab</sub> =	2737.50 lb		
A <sub>Pavement</sub> =	119.36 ft <sup>2</sup>	{Area of pavement}	
W <sub>Pavement</sub> =	16709.70 lb	{weight of pavement per linear foot of fill}	
W <sub>pavement +Slab</sub> =	19447.20 lb	{weight of pavement + Distribution Slab per linear foot of fill}	
δ=	45.00	{EPS/Concrete Friction Angle}	[NCHRP Table 3.2]
δ=	30.00	{EPS/EPS Friction Angle}	[NCHRP Table 3.2]
μ=	1.00	{EPS/Concrete coefficient of friction}	
μ=	0.58	{EPS/EPS coefficient of friction}	
Ø=	32.00	{Friction Angle of Backfill}	
ω=	-59.81	{slope of the back of the geofoam block along the block edges}	
i=	4.70	{Back of wall slope angle}	
δ=	16.00	{friction angle between soil and wall}	
K <sub>A</sub> =	0.0013	{active pressure coefficient}	[NCHRP Eq. 4.23]
P <sub>A</sub> =	39.15 lb	{active force}	[NCHRP Figure 4.29]
P <sub>AX</sub> =	28.25 lb	{Horizontal active force}	
P <sub>ES</sub> =	45.75 lb	{Uniform surcharge load force}	
P <sub>LS</sub> =	4.09 lb	{Live load surcharge force}	
PGA=	0.500 g	{Peak ground acceleration}	[GD Table 4]
Kh=	0.250	{horizontal seismic coefficient}	
Ψ=	14.036	{Angle of the resultant body force in the soil from vertical}	
K <sub>AE</sub> =	0.0750	{Earthquake active earth pressure coefficient}	[NCHRB Eq. 4.21]
P <sub>AE</sub> =	2213.10 lb	{seismic force}	[NCHRB Figure 4.29]
ΣP=	2252.24 lb	{P <sub>A</sub> +P <sub>AE</sub> }	
ΣPx=	1675.15 lb	{Horizontal component of ΣP}	
ΣPy=	1559.16 lb	{Vertical component of ΣP}	

## EPS/Distribution Slab Sliding Interface

γ <sub>p,EH</sub> =	1.00	{Earth horizontal load factor}	[AASHTO Table 3.4.1-1]
P <sub>A</sub> =	1.86 lb	{active force}	[NCHRP Figure 4.29]
γ <sub>p,ES</sub> =	1.00	{Uniform surcharge load factor}	[AASHTO Table 3.4.1-1]
P <sub>ES</sub> =	9.96 lb	{Uniform surcharge load force}	
γ <sub>p,LS</sub> =	0.50	{Live load surcharge load factor}	[AASHTO Table 3.4.1-1]
P <sub>LS</sub> =	0.76 lb	{Live load surcharge force}	
γ <sub>EQ</sub> =	1.00	{Earthquake load factor}	[AASHTO Table 3.4.1-1]
P <sub>AE</sub> =	154.12 lb	{seismic force}	[NCHRB Figure 4.29]
ΣP=	155.98 lb	{P <sub>A</sub> +P <sub>AE</sub> }	
ΣPx=	123.28 lb	{Horizontal component of ΣP}	
Ø <sub>r</sub> =	1.00	{Resistance factor}	
γ <sub>p,EV</sub> =	1.00	{Earth vertical load factor}	[AASHTO Table 3.4.1-1]
γ <sub>p,DC</sub> =	1.00	{Dead load load factor}	[AASHTO Table 3.4.1-1]
Sliding FS=	3.901	OK {Factor of safty >1.2}	

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*EPS/EPS Sliding Interface*

$H_{EPS} =$	2.00 ft	{EPS 1 layer height}	
$A_{EPS} =$	73.00 ft <sup>2</sup>	{Area of EPS fill}	
$W_{EPS} =$	131.40 lb	{weight of EPS fill per linear foot of fill}	
$\gamma_{p,EH} =$	1.00	{Earth horizontal load factor}	[AASHTO Table 3.4.1-1]
$P_A =$	3.69 lb	{active force}	[NCHRP Figure 4.29]
$\gamma_{p,ES} =$	1.00	{Uniform surcharge load factor}	[AASHTO Table 3.4.1-1]
$P_{ES} =$	14.05 lb	{Uniform surcharge load force}	
$\gamma_{p,LS} =$	0.50	{Live load surcharge load factor}	[AASHTO Table 3.4.1-1]
$P_{LS} =$	1.07 lb	{Live load surcharge force}	
$\gamma_{EQ} =$	1.00	{Earthquake load factor}	[AASHTO Table 3.4.1-1]
$P_{AE} =$	295.50 lb	{seismic force}	[NCHRP Figure 4.29]
$\Sigma P =$	299.20 lb	{ $P_A + P_{AE}$ }	
$\Sigma P_x =$	231.04 lb	{Horizontal component of $\Sigma P$ }	
$\phi_T =$	1.00	{Resistance factor}	
$\gamma_{p,EV} =$	1.00	{Earth vertical load factor}	[AASHTO Table 3.4.1-1]
$\gamma_{p,DC} =$	1.00	{Dead load load factor}	[AASHTO Table 3.4.1-1]
Sliding FS=	2.205	OK	{Factor of safety > 1.2}

[J] Horizontal Sliding due to collision force*EPS/Distribution Slab Sliding Interface*

$L_s =$	10000.00 lb	{Collision equivalent static load}	[BDM 10.3.2-2]
$L_{slab,min} =$	40.00 ft	{minimum slab length}	
$\gamma_{CT} =$	1.00	{Collision Force load factor}	[AASHTO Table 3.4.1-1]
$\Sigma P =$	262.06 lb	{ $\Sigma$ Horizontal component of PA}	
Sliding FS=	74.209	OK	{Factor of safety > 1.2}

*EPS/EPS Sliding Interface*

$\Sigma P =$	267.79 lb	{ $\Sigma$ Horizontal component of PA}
Sliding D/C=	41.928	OK {Factor of safety > 1.2}

[G] Load Bearing Capacity

$T_p =$	0.85 ft	{Pavement thickness}
$T_f =$	2.42 ft	{Fill Thickness @ CL Use for Load Bearing Check}
$T_{slab} =$	0.50 ft	{Distribution Slab thickness}
$\sigma_{pav} =$	0.83 ksf	{Gravity Stress due to Pavement Dead Load}
$\sigma_{fill} =$	2.02 ksf	{Gravity Stress due to Soil Dead Load}
$\sigma_{slab} =$	0.52 ksf	{Gravity Stress due to distribution slab Load}
$\sigma_{DL} =$	3.36 ksf	{Total Dead Load Gravity Stress}

*Live Load Stresses*

$I =$	0.300	{Impact Coefficient}
Design Truck	HS20-44	
Axle Loading =	32.00 kips	
$Q_D =$	16.00 kips	{Wheel Loads}
$Q_D =$	20.80 kips	{LL + Impact One Tire}

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*Estimation of Equivalent Traffic Stresses*

The chart shown below has been interpolated using Figure 6.17 NCHRP project 24-11 (shown in Metric). The equivalent stress on top of the assumed EPS block will shown below using this conversion.

Conversion Factors:

$$1\text{kPa} = 0.145 \text{ psi}$$

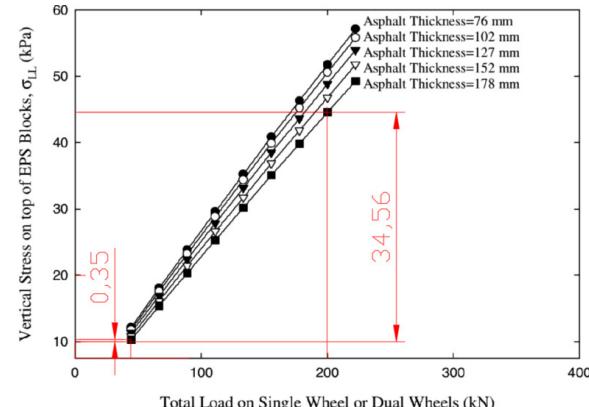
$$1\text{kN} = 224.800 \text{ pounds}$$

LL + Impact One Tire, $Q_D$ (pounds)	Vertical Stress @ Top of EPS, $\sigma_{LL}$ (psi)
10116	1.50
44960	6.46

Slope Ratio = 0.0001424 psi/kip

$$\sigma_{LL} = 435.13 \text{ lb/ft}^2$$

$$A_{CD} = 47.80 \text{ ft}^2$$



{Live Load Stress}

{circular contact area}

Note that these are empirical relationships based on SI units. Therefore, ACD must be converted from ft<sup>2</sup> to m<sup>2</sup>

$A_{CD} = 4.44 \text{ m}^2$	{circular contact area}
$L' = 2.91 \text{ m}$	
$L = 2.54 \text{ m}$	8.33 ft
$B = 1.75 \text{ m}$	5.74 ft

If the center-to-center wheel spacing on the design truck is  $\leq B$ , stress overlap will occur

$B_{overlap} = 9.74 \text{ ft}$	
Area = 81.15 ft <sup>2</sup>	
$\sigma_{LL,comb.} = 512.65 \text{ lb/ft}^2$	
FS = 1.200	
$\sigma_{total} = 8.31 \text{ psi}$	
Select EPS Type = EPS29	
$\sigma_e = 10.90 \text{ psi}$	{Maximum Elastic Stress Limit}
D/C = 0.762	[ASTM D6817]



## **RETAINING WALL 09.90L - A & B**

### **1.3 — *Vertical Wall Panel Design***

<b>PARSONS</b>		MADE BY	DATE	CHK BY	DATE
W. Metwally		11/12/2021		E. Kelley	
Job Number	WBS Number	TITLE	I-405 Renton to Bellevue		
650512	00531		Vertical Wall Panel Design		

**[A] BASIS**

The design of the precast wall panel.

**[B] REFERENCES**

Acronym	Source
AASHTO	AASHTO LRFD Bridge Design Specifications (8th Edition)
BDM	WSDOT Bridge Design Manual M23-50.18 (2018 Edition)

**[C] CALCULATION LEGEND**

-Standard cell format for User Input, Check Passed, No Good, and Output, respectively:

User Input	OK	NG	Output
------------	----	----	--------

-Curly brackets, { }, are used to enclose descriptions of inputs and calculations

-Square brackets, [ ], are used to enclose references

-All stations (##+##.##) have units 'ft.' Other units are noted with their respective values.

**[D] MATERIAL PROPERTIES**

$\gamma_{c,E} = 150 \text{ lb/ft}^3$	{reinforced concrete unit weight for capacity analysis}	[BDM Table 3.8-1]
$\gamma_c = 145 \text{ lb/ft}^3$		[AASHTO-Table 3.5.1-1]
$\gamma_{pav} = 140 \text{ lb/ft}^3$	{pavement unit weight}	[AASHTO-Table 3.5.1-1]
$\gamma_{soil} = 120 \text{ lb/ft}^3$	{soil unit weight}	
$\gamma_{EPS} = 1.80 \text{ lb/ft}^3$	{EPS unit weight}	[ASTM D6817]

**Unconfined Concrete**

$f'_c$	$K_1$	$E_c$	$f_r$	$\epsilon_{cu}$	$v_c$	$G_c$
4.0 ksi	1.00	4266.2 ksi	0.48 ksi	0.003	0.2	1777.6 ksi

$$f'_c = \{\text{concrete compressive strength}\}$$

$$E_c = 120,000 K_1 \gamma_c^{2/3} f'_c^{0.33}$$

$$f_r = 0.24\sqrt{(f'_c)} \quad \{\text{concrete tension capacity for this analysis}\}$$

$$\epsilon_{cu} = \{\text{maximum unconfined concrete strain}\}$$

$$v_c = \{\text{poisson's ratio for concrete}\}$$

$$G_c = E_c / (2*(1+v_c)) \quad \{\text{concrete shear modulus}\}$$

[AASHTO-5.4.2.4-1]

[AASHTO-5.4.2.6]

[AASHTO-5.6.2.1]

[AASHTO 5.4.2.5]

**ASTM A706 Grade 60 Reinforcing Steel**

$f_y$	$f_u$	$E_s$	$\epsilon_y$
60.0 ksi	80.0 ksi	29000 ksi	0.0021

$$f_y = \{\text{minimum yield strength}\}$$

$$f_u = \{\text{minimum tensile strength}\}$$

$$E_s = \{\text{steel modulus of elasticity}\}$$

$$\epsilon_y = \{\text{nominal yield strain}\}$$

[ASTM A706-16 Table A1.2]

[ASTM A706-16 Table A1.2]

[AASHTO 5.4.3.2]

<b>PARSONS</b>		MADE BY	DATE	CHK BY	DATE
Job Number	WBS Number	W. Metwally	11/12/2021	E. Kelley	11/21/2021
650512	00531	TITLE	I-405 Renton to Bellevue		
			Vertical Wall Panel Design		

[E] Precast Wall loads during construction

H <sub>panel</sub> =	18.00 ft	{Wall Panel height}
T <sub>panel</sub> =	0.77 ft	{Wall Panel thickness}
W <sub>DL</sub> =	0.12 klf	{Unfactored Vertical Dead Load}
Impact factor=	1.20	
γ <sub>p</sub> =	1.25	{STR I load factor}
M <sub>U,DL</sub> =	7.02 k-ft	{DL moment}
V <sub>U,DL</sub> =	1.56 kip	{DL Shear force}

[F] Wind load (WS):

V =	110.00 mph	{design 3-second gust wind speed }	[AASHTO Table 3.8.1.1.2-1]
Z=	23.67 ft		
KZ =	0.64	{pressure exposure and elevation coefficient}	[AASHTO Table C3.8.1.2.1-1]
G =	1.00	{gust effect factor}	[AASHTO Table 3.8.1.2.1-1]
CD =	1.20	{drag coefficient}	[AASHTO Table 3.8.1.2.1-2]
PZ =	0.024 ksf	{design wind pressure}	[AASHTO 3.8.1.2.1-1]
Fws=	0.43 kip	{Horizontal Wind Load}	
γ <sub>p</sub> =	1.00	{Strength III Load Factor}	
M <sub>u,ws</sub> =	1.91 k-ft	{WS moment}	
V <sub>u,ws</sub> =	0.21 kip	{DL Shear force}	

[G] Wall Design - Flexural Reinforcement

W <sub>wall</sub> =	12.00 in	{Wall width (assume section 1ft)}	
d=	8.00 in	{wall depth at face of the wall}	
C=	2.00 in	{concrete cover}	
bar#=	6		
d <sub>bar</sub> =	0.75 in	{bar diameter}	
A <sub>bar</sub> =	0.44 in <sup>2</sup>	{bar area}	
spacing=	9.00 in	{RFT Spacing}	
d <sub>s</sub> =	5.63 in	{depth from compression face of Wall to centroid of tensile reinforcement}	
M <sub>u</sub> =	7.02 k-ft	{Factored moment}	
Ø=	0.90		[AASHTO-5.6.2.2]
β <sub>1</sub> =	0.85		
ε <sub>cu</sub> =	0.003		[AASHTO-5.6.2.1]
ε <sub>tl</sub> =	0.005	{tension controlled steel strain minimum limit}	[AASHTO-5.6.2.1]
ε <sub>cl</sub> =	0.002	{compression controlled steel strain minimum limit}	[AASHTO-5.6.2.1]
As provided=	0.59 in <sup>2</sup>		
C=	1.01		[AASHTO-5.6.3.1.2-3]
ε <sub>t</sub> =	0.01	Tension [strain compatibility]	
a=	0.86		[AASHTO-5.6.3.2.2]
φM <sub>n</sub> =	13.71		[AASHTO-5.6.3.2.2]
D/C=	0.512		

<b>PARSONS</b>		MADE BY	DATE	CHK BY	DATE
Job Number	WBS Number	TITLE	11/12/2021	E. Kelley	11/21/2021
650512	00531	I-405 Renton to Bellevue Vertical Wall Panel Design			

Check minimum flexural reinforcement requirements.

[AASHTO-5.6.3.3.2]

fr =	0.48 ksi	= $0.24\sqrt{f_c}$	[AASHTO-5.4.2.6]
$\gamma_1$ =	1.6	{non-segmental}	[AASHTO-5.6.3.3.2]
$\gamma_2$ =	1.0	{no tendons}	[AASHTO-5.6.3.3.2]
$\gamma_3$ =	0.75	{A706 reinforcement}	[AASHTO-5.6.3.3.2]
$S_c$ =	0.07 ft <sup>3</sup>		[AASHTO-5.6.3.3.2]
$M_{cr}$ =	6.14 k-ft		[AASHTO-5.6.3.3.2-1]
$1.33M_{umax}$ =	9.34 k-ft		
$M_{rmin}$ =	6.14 k-ft		[AASHTO-5.6.3.3.2]
$\phi M_n$ =	13.71 k-ft		
D/C =	0.448		

I[H] SHEAR CAPACITY (STRENGTH)

$\beta$ =	2.00		
$b_v$ =	12.00 in	{Effective Web Width}	
$d_v$ =	5.76 in	{Effective Shear Depth max( 0.90d <sub>e</sub> or 0.72h)}	
Xcritical=	9.75 in	{The critical section for shear}	
$V_u$ =	1.56 kip		[AASHTO-5.7.3.3-3]
$V_c$ =	8.74 kip		[AASHTO-5.7.3.3-1]
$V_{nl}$ =	8.74 kip		[AASHTO-5.7.3.3-2]
$V_{n2}$ =	69.12 kip		[AASHTO-5.7.3.3]
$\phi_v V_n$ =	7.86 kip		
D/C=	0.199		

II Wall Design - Temperature & Shrinkage

$A_{s,min}$ =	0.11 in <sup>2</sup> /ft		[AASHTO-5.10.6]
bar#=	4		
dbar=	0.50 in	{bar diameter}	
Abar=	0.20 in <sup>2</sup>	{bar area}	
spacing=	12.00 in	{RFT Spacing}	
C=	2.00 in	{concrete cover}	
No. of bars=	1		
As req'd =	0.11 in <sup>2</sup>		
$A_{s,prov}$ =	0.20 in <sup>2</sup>		
D/C=	0.550		
$S_{max\ req}$ =	12.00 in	{max spacing requirements}	[AASHTO-5.10.6]
D/C=	1.000		



## **RETAINING WALL 09.90L - A & B**

### *1.4 — Grade Beam Design*

<b>PARSONS</b>		MADE BY	DATE	CHK BY	DATE
Job Number	WBS Number	W. Metwally	11/12/2021	E. Kelley	11/21/2021
650512	00531	TITLE	I-405 Renton to Bellevue		
			Grade Beam Design		

**[A] BASIS**

Design of the longitudinal grade beam is based only on the vertical dead load of the precast wall panel.  
LRFD Design criteria and applicable load factors will be used for the design of the grade beam.

**[B] REFERENCES**

Acronym	Source
AASHTO	AASHTO LRFD Bridge Design Specifications (8th Edition)
BDM	WSDOT Bridge Design Manual M23-50.18 (2018 Edition)

**[C] CALCULATION LEGEND**

-Standard cell format for User Input, Check Passed, No Good, and Output, respectively:

User Input	OK	NG	Output
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-Curly brackets, { }, are used to enclose descriptions of inputs and calculations

-Square brackets, [ ], are used to enclose references

-All stations (##+##.###) have units 'ft.' Other units are noted with their respective values.

**[D] MATERIAL PROPERTIES**

$\gamma_{c,E} = 150 \text{ lb/ft}^3$	{reinforced concrete unit weight for capacity analysis}	[BDM Table 3.8-1]
$\gamma_c = 145 \text{ lb/ft}^3$		[AASHTO-Table 3.5.1-1]
$\gamma_{pav} = 140 \text{ lb/ft}^3$	{pavement unit weight}	[AASHTO-Table 3.5.1-1]
$\gamma_{soil} = 120 \text{ lb/ft}^3$	{soil unit weight}	
$\gamma_{EPS} = 1.80 \text{ lb/ft}^3$	{EPS unit weight}	[ASTM D6817]

**Unconfined Concrete**

$f'_c$	$K_1$	$E_c$	$f_r$	$\varepsilon_{cu}$	$v_c$	$G_c$
4.0 ksi	1.00	4266.2 ksi	0.48 ksi	0.003	0.2	1777.6 ksi

$$f'_c = \{\text{concrete compressive strength}\}$$

$E_c = 120,000 K_1 \gamma_c^{2/3} f'_c^{0.33}$  [AASHTO-5.4.2.4-1]

$$f_r = 0.24 \sqrt{(f'_c)} \quad \{\text{concrete tension capacity for this analysis}\}$$

$$\varepsilon_{cu} = \{\text{maximum unconfined concrete strain}\}$$

$$v_c = \{\text{poisson's ratio for concrete}\}$$

$$G_c = E_c / (2 * (1 + v_c)) \quad \{\text{concrete shear modulus}\}$$

[AASHTO-5.4.2.4-1]

[AASHTO-5.4.2.6]

[AASHTO-5.6.2.1]

[AASHTO 5.4.2.5]

**ASTM A706 Grade 60 Reinforcing Steel**

$f_y$	$f_u$	$E_s$	$\varepsilon_y$
60.0 ksi	80.0 ksi	29000 ksi	0.0021

$$f_y = \{\text{minimum yield strength}\}$$

[ASTM A706-16 Table A1.2]

$$f_u = \{\text{minimum tensile strength}\}$$

[ASTM A706-16 Table A1.2]

$$E_s = \{\text{steel modulus of elasticity}\}$$

[AASHTO 5.4.3.2]

$$\varepsilon_y = \{\text{nominal yield strain}\}$$

<b>PARSONS</b>		MADE BY	DATE	CHK BY	DATE
Job Number	WBS Number	W. Metwally	11/12/2021	E. Kelley	11/21/2021
650512	00531	TITLE	I-405 Renton to Bellevue		
			Grade Beam Design		

[E] Precast Wall Geometry

H <sub>panel</sub> =	18.00 ft	{Wall Panel height}
T <sub>panel</sub> =	0.77 ft	{Wall Panel thickness}
e <sub>wall</sub> =	0.15 ft	{Wall eccentricity}
W=	2.08 klf	{Unfactored Vertical Load on Grade Beam}
H <sub>footing</sub> =	1.17 ft	{Footing height}
W <sub>footing</sub> =	2.00 ft	{Footing width}
W=	0.35 klf	{Unfactored Vertical Load on Grade Beam}
W <sub>total</sub> =	2.43 klf	

[F] Soil Pressure Check

M=	0.30 k-ft	{moment at the CL of the footing}
e=	0.12 ft	{eccentricity}
b/6=	0.33 ft	OK
q <sub>max</sub> =	1.67 ksf	{Actual Soil Pressure}
q <sub>a</sub> =	3.00 ksft	OK {Allowable Soil Pressure}

[G] Grade Beam Design - Flexural Reinforcement

W <sub>footing</sub> =	24.00 in	{Footing width}
d=	10.00 in	{Footing depth at face of the wall}
C=	3.00 in	{concrete cover}
bar#=	4	
dbar=	0.50 in	{bar diameter}
Abar=	0.20 in <sup>2</sup>	{bar area}
spacing=	12.00 in	{RFT Spacing}
ds=	6.75 in	{depth from compression face of beam to centroid of tensile reinforcement}
Xcritical=	13.75 in	{The critical section for moment}
M=	1.10 k-ft	{unfactored moment}
γ <sub>p</sub> =	1.50	
M <sub>u</sub> =	1.65 k-ft	{Factored moment}
∅=	0.90	
β <sub>1</sub> =	0.85	[AASHTO-5.6.2.2]
ε <sub>cu</sub> =	0.003	
ε <sub>tl</sub> =	0.005	{tension controlled steel strain minimum limit} [AASHTO-5.6.2.1]
ε <sub>cl</sub> =	0.002	{compression controlled steel strain minimum limit} [AASHTO-5.6.2.1]
As provided=	0.20 in <sup>2</sup>	
C=	0.35	[AASHTO-5.6.3.1.2-3]
ε <sub>t</sub> =	0.06	
a=	0.29	[AASHTO-5.6.3.2.2]
∅M <sub>n</sub> =	5.94	[AASHTO-5.6.3.2.2]
D/C=	0.277	
Tension	[strain compatibility]	

<b>PARSONS</b>		MADE BY	DATE	CHK BY	DATE
Job Number	WBS Number	W. Metwally	11/12/2021	E. Kelley	11/21/2021
Job Number	WBS Number	TITLE	I-405 Renton to Bellevue		
650512	00531		Grade Beam Design		

Check minimum flexural reinforcement requirements.

[AASHTO-5.6.3.3.2]

$f_r =$	0.48 ksi	$= 0.24\sqrt{f_c}$	[AASHTO-5.4.2.6]
$\gamma_1 =$	1.6	{non-segmental}	[AASHTO-5.6.3.3.2]
$\gamma_2 =$	1.0	{no tendons}	[AASHTO-5.6.3.3.2]
$\gamma_3 =$	0.75	{A706 reinforcement}	[AASHTO-5.6.3.3.2]
$S_c =$	0.12 ft <sup>3</sup>		[AASHTO-5.6.3.3.2]
$M_{cr} =$	9.60 k-ft		[AASHTO-5.6.3.3.2-1]
$1.33M_{umax} =$	2.19 k-ft		
$M_{rmin} =$	2.19 k-ft		[AASHTO-5.6.3.3.2]
$\phi M_n =$	5.94 k-ft		
$D/C =$	0.368		

I[H] SHEAR CAPACITY (STRENGTH)

$\beta =$	2.00		
$b_v =$	12.00 in	{Effective Web Width}	
$d_v =$	7.20 in	{Effective Shear Depth max( 0.90d <sub>e</sub> or 0.72h)}	
$X_{critical} =$	9.75 in	{The critical section for shear}	
$V_u =$	1.36 kip		
$V_c =$	10.92 kip		[AASHTO-5.7.3.3-3]
$V_{n1} =$	10.92 kip		[AASHTO-5.7.3.3-1]
$V_{n2} =$	86.40 kip		[AASHTO-5.7.3.3-2]
$\phi_v V_n =$	9.83 kip		[AASHTO-5.7.3.3]
$D/C =$	0.138		

II Grade Beam Design - Temperature & Shrinkage

$A_{s min} =$	0.11 in <sup>2</sup> /ft		[AASHTO-5.10.6]
bar# =	4		
dbar =	0.50 in	{bar diameter}	
Abar =	0.20 in <sup>2</sup>	{bar area}	
spacing =	9.00 in	{RFT Spacing}	
C =	2.00 in	{side concrete cover}	
No. of bars =	3		
$A_{s req'd} =$	0.07 in <sup>2</sup>		
$A_{s, prov} =$	0.20 in <sup>2</sup>		
$D/C =$	0.367		
$S_{max\ req} =$	12.00 in	{max spacing requirements}	[AASHTO-5.10.6]
	0.750		



## **RETAINING WALL 09.90L - A & B**

### **1.5 — *Distribution Slab Design***

<b>PARSONS</b>		MADE BY W. Metwally	DATE 11/12/2021	CHK BY E. Kelley	DATE 11/21/2021
Job Number 650512	WBS Number 00531	TITLE	I-405 Renton to Bellevue Distribution Slab Design		

[A] BASIS

The distribution slab will be designed as a slab on grade. It will be modeled in CSIBridge as a 30'x36.5' plate structure with 1'x1' max grids.

[B] REFERENCES

Acronym	Source
AASHTO	AASHTO LRFD Bridge Design Specifications (8th Edition)
BDM	WSDOT Bridge Design Manual M23-50.18 (2018 Edition)

[C] CALCULATION LEGEND

-Standard cell format for User Input, Check Passed, No Good, and Output, respectively:

User Input	OK	NG	Output
------------	----	----	--------

-Curly brackets, { }, are used to enclose descriptions of inputs and calculations

-Square brackets, [ ], are used to enclose references

-All stations (###+##.##) have units 'ft.' Other units are noted with their respective values.

[D] MATERIAL PROPERTIES

$\gamma_{c,E} =$	150 lb/ft <sup>3</sup>	{reinforced concrete unit weight for capacity analysis}	[BDM Table 3.8-1]
$\gamma_{c,E} =$	155 lb/ft <sup>3</sup>	{reinforced concrete unit weight for the model}	[AASHTO-Table 3.5.1-1]
$\gamma_c =$	145 lb/ft <sup>3</sup>		[AASHTO-Table 3.5.1-1]
$\gamma_{pav} =$	140 lb/ft <sup>3</sup>	{pavement unit weight}	[AASHTO-Table 3.5.1-1]
$\gamma_{soil} =$	120 lb/ft <sup>3</sup>	{soil unit weight}	
$\gamma_{EPS} =$	1.80 lb/ft <sup>3</sup>	{EPS unit weight}	[ASTM D6817]

**Unconfined Concrete**

$f'_c$	$K_1$	$E_c$	$E_c$	$f_r$	$\varepsilon_{cu}$	$v_c$	$G_c$
4.0 ksi	1.00	4266.2 ksi	4555.4 ksi	0.48 ksi	0.003	0.2	1777.6 ksi

$f'_c =$  {concrete compressive strength }

$E_c = 120,000K_1\gamma_c^2 f'_c^{0.33}$

$f_r = 0.24\sqrt{f'_c}$  {concrete tension capacity for this analysis}

$\varepsilon_{cu} =$  {maximum unconfined concrete strain}

$v_c =$  {poisson's ratio for concrete}

$G_c = E_c / (2*(1+v_c))$  {concrete shear modulus}

[AASHTO-5.4.2.4-1]

[AASHTO-5.4.2.6]

[AASHTO-5.6.2.1]

[AASHTO 5.4.2.5]

**EPS**

Eti
1.218 ksi

Eti= {Initial Secant Young's Modulus}

[NCHRP table f.2]

**ASTM A706 Grade 60 Reinforcing Steel**

$f_y$	$f_u$	$E_s$	$\varepsilon_y$
60.0 ksi	80.0 ksi	29000 ksi	0.0021

$f_y =$  {minimum yield strength}

[ASTM A706-16 Table A1.2]

$f_u =$  {minimum tensile strength}

[ASTM A706-16 Table A1.2]

$E_s =$  {steel modulus of elasticity}

[AASHTO 5.4.3.2]

$\varepsilon_y =$  {nominal yield strain}

<b>PARSONS</b>		MADE BY W. Metwally	DATE 11/12/2021	CHK BY E. Kelley	DATE 11/21/2021
Job Number 650512	WBS Number 00531	TITLE	I-405 Renton to Bellevue Distribution Slab Design		

[E] Model inputs:

Ks=	101.50 pci	{Approximate Modulus of Subgrade Reaction}
CS =	14.62 kips/in	{Compression Spring}

Load Combination STR  
 $1.25 \times DC + 1.35 \times EV + 1.75 \times LL$

Load Combination SER  
 $1.00 \times DC + 1.00 \times EV + 1.00 \times LL$

H <sub>fill(MIN)</sub> =	2.17 ft	{Minimum fill height above the EPS}
H <sub>fill(MAX)</sub> =	4.37 ft	{Maximum fill height above the EPS}
W <sub>top</sub> =	36.50 ft	{EPS width at top}
L <sub>EPS</sub> =	30.00 ft	{EPS length}
H <sub>pav. Avg</sub> =	3.27 ft	{Average fill height above the EPS}
W <sub>Pavement</sub> =	0.46 ksf	{weight of pavement per sq. foot}

## Traffic Live Loads

Axle Loading =	32.00 kip	
Q <sub>D</sub> =	16.00 kip	{Wheel Loads}
I =	0.33	{Impact Coefficient}
Live Point Loads =	21.28 kip	{LL + Impact One Tire}
L <sub>T</sub> =	10.00 in	{Length of Tire Contact}
W <sub>T</sub> =	20.00 in	{Width of Tire Contact}
Distribution Factor =	1.15	{Distribution Factor}
H =	3.27 ft	{Depth of Fill}
LL Distribution Factor =	3.76	
L <sub>TE</sub> =	4.59 ft	{Equivalent Length of Contact}
L <sub>WE</sub> =	5.43 ft	{Equivalent Width of Contact}
A <sub>AE</sub> =	24.93 ft <sup>2</sup>	{Equivalent Contact Area}
Distributed Live Load =	0.85 ksf	

[F] Model output:

M <sub>U,DL</sub> =	2.70 k-ft/ft	{STR moment}
V <sub>U,DL</sub> =	1.27 kip/ft	{STR Shear force}

<b>PARSONS</b>		MADE BY W. Metwally	DATE 11/12/2021	CHK BY E. Kelley	DATE 11/21/2021
Job Number 650512	WBS Number 00531	TITLE	I-405 Renton to Bellevue Distribution Slab Design		

[G] Slab Design - Flexural Reinforcement

W <sub>slab</sub> =	12.00 in	{Slab width (assume section 1ft)}
d=	6.00 in	{wall depth at face of the wall}
C=	3.00 in	{concrete cover}
bar#=	5	
dbar=	0.63 in	{bar diameter}
Abar=	0.31 in <sup>2</sup>	{bar area}
spacing=	7.00 in	{RFT Spacing}
ds=	2.69 in	{depth from compression face of beam to centroid of tensile reinforcement}
Mu=	2.70 k-ft	{Factored moment}
Ø=	0.90	
β <sub>1</sub> =	0.85	[AASHTO-5.6.2.2]
ε <sub>cu</sub> =	0.003	
ε <sub>tl</sub> =	0.005	[AASHTO-5.6.2.1]
ε <sub>cl</sub> =	0.002	[AASHTO-5.6.2.1]
As provided=	0.53 in <sup>2</sup>	
C=	0.92	[AASHTO-5.6.3.1.2-3]
ε <sub>t</sub> =	0.01	
a=	0.78	[AASHTO-5.6.3.2.2]
ØM <sub>n</sub> =	5.49	[AASHTO-5.6.3.2.2]
D/C=	0.492	

Check minimum flexural reinforcement requirements. [AASHTO-5.6.3.3.2]

fr =	0.48 ksi	= 0.24√(f <sub>c</sub> )	[AASHTO-5.4.2.6]
γ <sub>1</sub> =	1.6	{non-segmental}	[AASHTO-5.6.3.3.2]
γ <sub>2</sub> =	1.0	{no tendons}	[AASHTO-5.6.3.3.2]
γ <sub>3</sub> =	0.75	{A706 reinforcement}	[AASHTO-5.6.3.3.2]
S <sub>c</sub> =	0.04 ft <sup>3</sup>		[AASHTO-5.6.3.3.2]
M <sub>cr</sub> =	3.46 k-ft		[AASHTO-5.6.3.3.2-1]
1.33M <sub>umax</sub> =	3.59 k-ft		
M <sub>min</sub> =	3.46 k-ft		[AASHTO-5.6.3.3.2]
ØM <sub>n</sub> =	5.49 k-ft		
D/C=	0.629		

[H] SHEAR CAPACITY (STRENGTH)

β=	2.00	
b <sub>v</sub> =	12.00 in	{Effective Web Width}
d <sub>v</sub> =	4.32 in	{Effective Shear Depth max( 0.90d <sub>e</sub> or 0.72h)}
Xcritical=	9.75 in	{The critical section for shear}
V <sub>u</sub> =	1.27 kip	
V <sub>c</sub> =	6.55 kip	[AASHTO-5.7.3.3-3]
V <sub>n1</sub> =	6.55 kip	[AASHTO-5.7.3.3-1]
V <sub>n2</sub> =	51.84 kip	[AASHTO-5.7.3.3-2]
Ø <sub>v</sub> V <sub>n</sub> =	5.90 kip	[AASHTO-5.7.3.3]
D/C=	0.215	

<b>PARSONS</b>		MADE BY W. Metwally	DATE 11/12/2021	CHK BY E. Kelley	DATE 11/21/2021
Job Number 650512	WBS Number 00531	TITLE	I-405 Renton to Bellevue Distribution Slab Design		

[II] Slab Design - Temperature & Shrinkage

$A_s \text{ min} =$	0.11 in <sup>2</sup> /ft	[AASHTO-5.10.6]
bar# =	5	
dbar =	0.63 in	{bar diameter}
Abar =	0.31 in <sup>2</sup>	{bar area}
spacing =	7.00 in	{RFT Spacing}
C =	3.00 in	{side concrete cover}
No. of bars =	2	
As req'd =	0.06 in <sup>2</sup>	
A <sub>s, prov</sub> =	0.31 in <sup>2</sup>	
D/C =	0.177	
S <sub>max req</sub> =	12.00 in	{max spacing requirements}
	0.583	[AASHTO-5.10.6]

[J] Minimum Reinforcement

$A_{smin} =$	0.36 in <sup>2</sup>	{0.5% Ag}	[ACI 350 G.4.2]
$A_{s, prov} =$	0.53 in <sup>2</sup>		
D/C =	0.677		

[K] INTERFACE SHEAR BETWEEN MOMENT SLAB AND DISTRIBUTION SLAB

[AASHTO 5.7.4.]

For normal weight concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25"

c =	0.24 ksi	{cohesion factor}	[AASHTO 5.7.4.4]
$\mu =$	1	{friction factor}	
b <sub>bar</sub> =	12.00 in	{1' strip width}	
w <sub>bar</sub> =	40.31 in	{Conservative assume 0.5 length of the Moment slab}	
A <sub>cv</sub> =	483.75 in <sup>2</sup>	{concrete interface area}	
K1 =	0.25		
K2 =	1.50 ksi		
A <sub>vf, prov</sub> =	0.00 in <sup>2</sup> /ft		
P <sub>c</sub> =	0.00 k	{permanent net compressive force normal to shear plane = barrier self weight}	
V <sub>ni</sub> =	$cA_{cv} + \mu (A_{vf}f_y + P_c)$		[AASHTO 5.7.43]
V <sub>ni</sub> =	116.10 k	{nominal shear resistance}	
V <sub>ui</sub> =	16.90 k	{shear demand = transverse collision forces}	
D/C =	0.15	OK	



## **RETAINING WALL 09.90L - A & B**

*2.0 — Moment Slab Barrier Design*



## RETAINING WALL 09.90L - A & B

*2.1 — Typical Moment Slab Barrier Design*

Sensitive	<b>PARSONS</b>	MADE BY	DATE	CHK BY	DATE
		K. Garcia	9/15/2021	W. Metwally	11/17/2021
Job Number	WBS Number	TITLE		I-405 Renton to Bellevue	
649974	00531			Moment Slab Barrier	

**[A] BASIS**

- To check the Barrier Moment Slab Design supported by SE wall.

**[B] REFERENCES**

Acronym	Source
AASHTO	AASHTO LRFD Bridge Design Specifications (8th Edition)
BDM	WSDOT Bridge Design Manual M23-50.18 (2018 Edition)

**[C] CALCULATION LEGEND**

- Standard cell format for User Input, Check Passed, No Good, and Output, respectively:

User Input	OK	NG	Output
------------	----	----	--------

- Curly brackets, { }, are used to enclose descriptions of inputs and calculations

- Square brackets, [ ], are used to enclose references

- All stations (###+##.##) have units 'm.' Other units are noted with their respective values.

**[D] MATERIAL PROPERTIES**

$\gamma_{c,E}$ =	0.150 kcf
$\gamma_c$ =	0.145 kcf
$\gamma_{pav}$ =	0.140 kcf
$\gamma_{soil}$ =	0.120 kcf

{reinforced concrete unit weight for capacity analysis}

[BDM Table 3.8-1]

{reinforced concrete unit weight}

[AASHTO-Table 3.5.1-1]

{pavement unit weight}

[AASHTO-Table 3.5.1-1]

{soil unit weight}

**Unconfined Concrete**

$f_c$	$K_1$	$E_c$	$f_r$	$\epsilon_{cu}$	$v_c$	$G_c$
4.0 ksi	1.00	4266.2 ksi	0.48 ksi	0.003	0.2	1777.6 ksi

$f_c$  = {concrete compressive strength}

[AASHTO-5.4.2.4-1]

$E_c$  =  $= 120,000 K_1 \gamma_c^2 f_c^{0.33}$

[AASHTO-5.4.2.6]

$f_r$  =  $= 0.24\sqrt{f_c}$  {concrete tension capacity for this analysis}

[AASHTO-5.6.2.1]

$\epsilon_{cu}$  = {maximum unconfined concrete strain}

[AASHTO 5.4.2.5]

$v_c$  = {poisson's ratio for concrete}

[AASHTO 5.4.2.5]

$G_c$  =  $E_c / (2*(1+v_c))$  {concrete shear modulus}

**ASTM A706 Grade 60 Reinforcing Steel**

$f_y$	$f_u$	$E_s$	$\epsilon_y$
60.0 ksi	80.0 ksi	29000 ksi	0.0021

$f_y$  = {minimum yield strength}

[ASTM A706-16 Table A1.2]

$f_u$  = {minimum tensile strength}

[ASTM A706-16 Table A1.2]

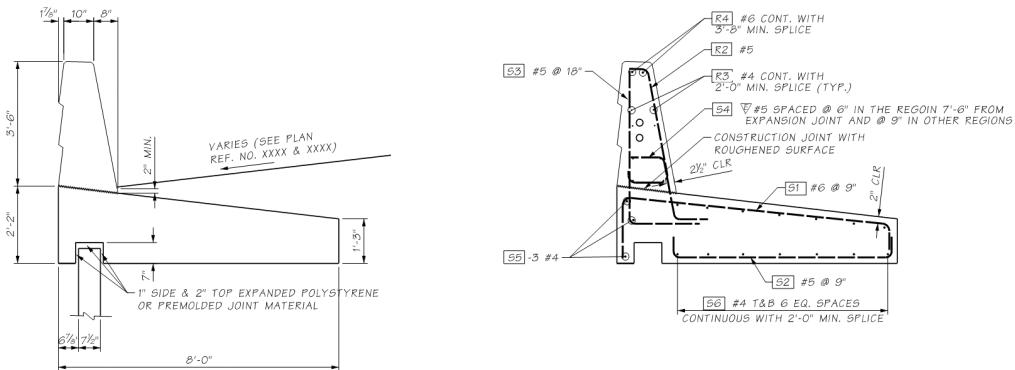
$E_s$  = {steel modulus of elasticity}

[AASHTO 5.4.3.2]

$\epsilon_y$  = {nominal yield strain}

Sensitive	<b>PARSONS</b>	MADE BY	DATE	CHK BY	DATE
	K. Garcia	9/15/2021		W. Metwally	11/17/2021
Job Number	WBS Number	TITLE	I-405 Renton to Bellevue	Moment Slab Barrier	

### [E] SLAB GEOMETRY AND REINFORCEMENT



#### 1. Barrier and Slab Geometry

Slab:

$t_{slab,min} =$	15.00 in	{min. slab thickness}
$t_{slab,max} =$	26.00 in	{max. slab thickness}
cover =	2.00 in	{concrete cover}
$b_{side,gap} =$	1.00 in	{side gap width @ notch}
$b_{top,gap} =$	2.00 in	{top gap height @ notch}
$b_{wall} =$	7.50 in	{wall thickness}
$b_{notch} =$	9.50 in	{total notch width}
$t_{notch} =$	7.00 in	{total notch height}
$b_{OHLab} =$	5.88 in	{slab edge to wall edge}
$b_{slab} =$	8.00 ft	{slab width}
$L_{slab,min} =$	40.00 ft	{minimum slab length}
$L_{slab,max} =$	120.00 ft	{maximum slab length}
$A_{slab} =$	1901.50 in <sup>2</sup>	{slab cross-sectional area}
$V_{slab} =$	528.19 ft <sup>3</sup>	{slab volume}

Barrier:

$h_{barrier} =$	42.00 in	{barrier height total}
$b_{bar,top} =$	10.00 in	{barrier width @ top}
$b_{bar,bot} =$	19.88 in	{barrier width @ bottom}
$b_{finish1} =$	1.88 in	{barrier finish width}
$b_{finish2} =$	1.50 in	{barrier finish width}
$h_{finish} =$	1.00 in	{barrier finish height}
$h_{seg} =$	14.00 in	{barrier finish segment height}
$A_{bar} =$	657.13 in <sup>2</sup>	{barrier cross-sectional area}
$V_{bar} =$	182.54 ft <sup>3</sup>	{barrier volume}

#### Check Max and Min Geometry:

$b_{slab,min,req} =$	4.00 ft	OK	{required minimum slab width}
$t_{slab,avg,req} =$	0.83 ft	OK	{required average slab depth}

[BDM 10.3.2-3]

[BDM 10.3.2-3]

#### Check Torsional Rigidity for Max Slab Length:

$2a =$	96.00 in	{total width of slab}
$2b =$	20.50 in	{average depth of slab}
$a =$	48.00 in	
$b =$	10.25 in	
$J =$	238602.31 in <sup>4</sup>	
$J_{60} =$	13900.17 in <sup>4</sup>	
$L(max) =$	120.00 ft	OK

$$J = a \cdot b^3 \left[ \frac{16}{3} - 3.36 \left( \frac{b}{a} \right) \left( 1 - \frac{b^4}{12a^4} \right) \right]$$

[BDM 10.3.2-3]

#### 2. Barrier Reinforcement

Bar Mark	Bar Size	$d_{b,L}$	$A_{b,L}$	$N_{b,L}$	
R4	#6	0.75 in	0.44 in <sup>2</sup>	2	{continuous}
R3	#4	0.50 in	0.20 in <sup>2</sup>	8	{continuous}

Bar Mark	Bar Size	$d_{b,V}$	$A_{b,V}$	$s_{b,V}$
R2	#5	0.63 in	0.31 in <sup>2</sup>	9.00 in
S3	#5	0.63 in	0.31 in <sup>2</sup>	18.00 in
S4	#5	0.63 in	0.31 in <sup>2</sup>	9.00 in

Sensitive	<b>PARSONS</b>	MADE BY	DATE	CHK BY	DATE
		K. Garcia	9/15/2021	W. Metwally	11/17/2021
Job Number	WBS Number	TITLE		I-405 Renton to Bellevue	
649974	00531			Moment Slab Barrier	

### 3. Slab Reinforcement

Bar Mark	Bar Size	d <sub>b,L</sub>	A <sub>b,L</sub>	N <sub>b,L</sub>	s <sub>b,L</sub>	
S5	#4	0.50 in	0.20 in <sup>2</sup>	3	-	{continuous}
S6 Top	#4	0.50 in	0.20 in <sup>2</sup>	7	12.15 in	{continuous}
S6 Bot	#4	0.50 in	0.20 in <sup>2</sup>	7	12.48 in	{continuous}

Bar Mark	Bar Size	d <sub>b,V</sub>	A <sub>b,V</sub>	s <sub>b,V</sub>
S1	#6	0.75 in	0.44 in <sup>2</sup>	9.00 in
S2	#5	0.63 in	0.31 in <sup>2</sup>	9.00 in

### 4. Check Maximum Spacing of Reinforcement

[AASHTO 5.10.3.2]

s <sub>b,L</sub>	s <sub>max</sub>	Check	s <sub>b,V</sub>	s <sub>max</sub>	Check
12.15 in.	18.00 in.	OK	9.00 in.	18.00 in.	OK
12.48 in.	18.00 in.	OK	9.00 in.	18.00 in.	OK

### [F] LOADING

#### Dead Loads

Slab Weight		Barrier	
V <sub>slab</sub>	W <sub>slab</sub>	V <sub>bar</sub>	W <sub>bar</sub>
528.19 ft <sup>3</sup>	<b>76.59 k</b>	182.54 ft <sup>3</sup>	<b>26.47 k</b>

#### Soil Weight Above Slab

	MIN SLOPE	MAX SLOPE	
d <sub>min</sub> =	0.19 ft	0.19 ft	{depth of roadway above slab @ face of barrier}
slope =	-8.00%	8.00%	{roadway slope}
t <sub>slab,bar</sub> =	1.98 ft	1.98 ft	{moment slab depth at face of barrier}
d <sub>max</sub> =	0.41 ft	1.42 ft	{depth of roadway above slab @ edge of slab}
A <sub>ws</sub> =	1.90 ft <sup>2</sup>	5.12 ft <sup>2</sup>	{area of wearing surface}
V <sub>ws</sub> =	75.99 ft <sup>3</sup>	204.77 ft <sup>3</sup>	{volume of wearing surface above slab}
w <sub>ws</sub> =	0.14 kcf	0.14 kcf	{wearing surface unit weight}
W <sub>ws</sub> =	10.64 k	28.67 k	

[BDM Table 3.8-1]

#### Wind Loads

-perpendicular to barrier face controls transverse loading

Wind Exposure Category B

[BDM 3.11.1]

$$P_z = 2.56 \times 10^{-6} V^2 K_z G C_D$$

$$Z = \boxed{33.00 \text{ ft}} \quad \{\text{height}\}$$

[AASHTO Table 3.8.1.1.2-1]

	STR III	SER I	
V =	110.00 mph	70.00 mph	{gust wind speed}
K <sub>z</sub> =	0.71	1.0	{pressure exposure and elevation coefficient}
G =	1.0	1.0	{gust effect factor}
C <sub>D</sub> =	1.2	1.2	{drag coefficient}
P <sub>z</sub> =	0.026 ksf	0.015 ksf	

[AASTHO 3.8.1.2.1-2]

	STRIII	SERI	
w <sub>T,b</sub> =	0.092 klf	0.053 klf	{barrier total transverse uniform wind line load}
z <sub>cg,WS,b</sub> =	1.75 ft	1.75 ft	{centroid of barrier wind load above point of rotation}
F <sub>WS,T</sub> =	0.092 k	0.053 k	{barrier total transverse wind load}
M <sub>WS,T</sub> =	0.52 k-ft/ft	0.30 k-ft/ft	{transverse uniform WS moment about POR}

Sensitive	<b>PARSONS</b>	MADE BY	DATE	CHK BY	DATE
		K. Garcia	9/15/2021	W. Metwally	11/17/2021
Job Number	WBS Number	TITLE		I-405 Renton to Bellevue	
649974	00531			Moment Slab Barrier	

### Vehicular Collision Forces

#### TL-4 Design Criteria for Structure Design

F <sub>t</sub>	F <sub>L</sub>	F <sub>v</sub>	L <sub>t</sub>	L <sub>L</sub>	L <sub>v</sub>	H <sub>e</sub> (min)
54.00 k	18.00 k	18.00 k	3.50 ft.	3.50 ft.	18.00 ft.	32.00 in.

F<sub>t</sub> = {Vehicle Impact Loading, Transverse}

[AASHTO Table A13.2-1]

F<sub>L</sub> = {Vehicle Impact Loading, Longitudinal}

[AASHTO Table A13.2-1]

F<sub>v</sub> = {Vehicle Impact Loading, Vertical}

[AASHTO Table A13.2-1]

L<sub>t</sub> = Longitudinal length of distribution of impact force F<sub>t</sub> along the railing

[AASHTO Table A13.2-1]

L<sub>L</sub> = Longitudinal length of distribution of friction force F<sub>L</sub>

[AASHTO Table A13.2-1]

L<sub>v</sub> = Longitudinal distribution of vertical force F<sub>v</sub> on top of railing

[AASHTO Table A13.2-1]

H<sub>e</sub> (min) = Height of wall (minimum)

[AASHTO Table A13.2-1]

#### TL-4 Design Criteria for Global Stability

$$L_s = \boxed{10.00 \text{ k}}$$

{equivalent static load}

[BDM 10.3.2-2]

$$h_a = \boxed{68.00 \text{ in.}}$$

{moment arm = top of barrier to point of rotation}

F <sub>OT,u</sub>	x <sub>OT,u</sub>	M <sub>OT,u</sub>
10.00 k	68.00 in	<b>56.7 k-ft</b>

$$x_{OT,u} = h_a$$

{Moment Arm}

$$M_{OT,u} = F_t * x_{OT,u}$$

{Vehicle Overturning Moment}

#### [G] GLOBAL STABILITY CHECK

##### Check Sliding Stability

W <sub>total</sub>	γ <sub>soil</sub>	δ <sub>soil</sub>	γ	R <sub>slide</sub>	F <sub>slide</sub>
113.69 k	0.120 kcf	32.0 °	1	<b>71.04 k</b>	<b>10.00 k</b>

γ<sub>soil</sub> = {unit weight, granular base material}

δ<sub>soil</sub> = {Friction Angle, granular base material}

R<sub>slide</sub> = W<sub>total</sub>\*tan(δ<sub>soil</sub>) {Frictional resistance to sliding}

F<sub>slide</sub> = √F<sub>t</sub><sup>2</sup>+F<sub>L</sub><sup>2</sup> {sliding force on Moment Slab}

γ = {load factor}

[BDM 10.3.2-B-4]

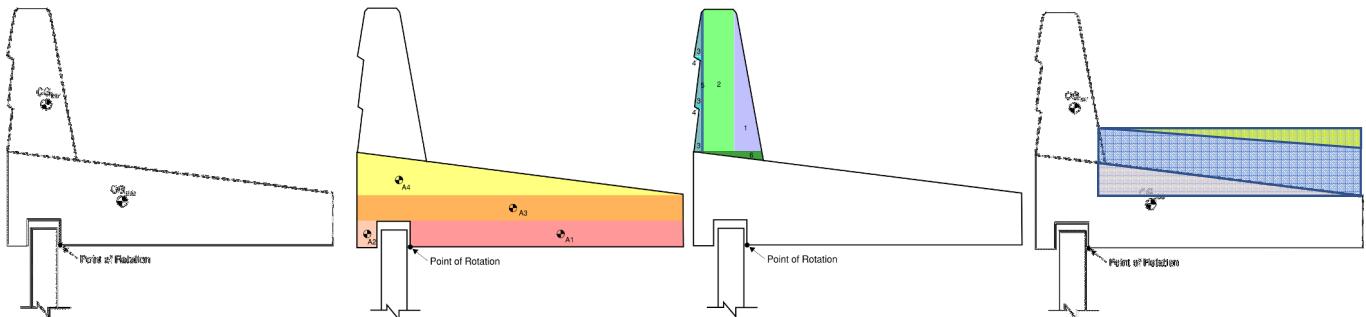
ϕ <sub>ext</sub>	ϕ <sub>ext</sub> R <sub>slide</sub>	D / C	Check
0.8	56.8 k	0.176	<b>OK</b>

ϕ<sub>ext</sub> = {Extreme Limit State}

[BDM 10.3.2-B-4]

Sensitive	<b>PARSONS</b>	MADE BY	DATE	CHK BY	DATE
	K. Garcia	9/15/2021		W. Metwally	11/17/2021
Job Number	WBS Number	TITLE	I-405 Renton to Bellevue		
649974	00531		Moment Slab Barrier		

**Check Overturning Stability**



Slab CG			
Dimensions	Area	$x_{cg}$	
$b_{A1} = 80.63 \text{ in}$ $h_{A1} = 7.00 \text{ in}$	$564.38 \text{ in}^2$	3.36 ft	
$b_{A2} = 5.88 \text{ in}$ $h_{A2} = 7.00 \text{ in}$	$41.13 \text{ in}^2$	-1.04 ft	
$b_{A3} = 96.00 \text{ in}$ $h_{A3} = 8.00 \text{ in}$	$768.00 \text{ in}^2$	2.72 ft	
$b_{A4} = 96.00 \text{ in}$ $h_{A4} = 11.00 \text{ in}$	$528.00 \text{ in}^2$	1.39 ft	
<b>TOTAL =</b>	<b><math>1901.50 \text{ in}^2</math></b>	<b>2.46 ft</b>	
	$M_{R,slab}$	<b>188.2 k-ft</b>	

Barrier CG			
Dimensions	Area	$x_{cg}$	
$b_{A1} = 8.00 \text{ in}$ $h_{A1} = 42.00 \text{ in}$	$168.00 \text{ in}^2$	-0.07 ft	
$b_{A2} = 10.00 \text{ in}$ $h_{A2} = 42.00 \text{ in}$	$420.00 \text{ in}^2$	-0.71 ft	
$b_{A3} = 1.50 \text{ in}$ $h_{A3} = 13.00 \text{ in}$	$29.25 \text{ in}^2$	-1.20 ft	
$b_{A4} = 1.50 \text{ in}$ $h_{A4} = 1.00 \text{ in}$	$1.50 \text{ in}^2$	-1.20 ft	
$b_{A5} = 0.38 \text{ in}$ $h_{A5} = 42.00 \text{ in}$	$15.75 \text{ in}^2$	-1.14 ft	
$b_{A6} = 19.88 \text{ in}$ $h_{A6} = 2.28 \text{ in}$	$22.63 \text{ in}^2$	-0.18 ft	
<b>TOTAL =</b>	<b><math>657.13 \text{ in}^2</math></b>	<b>-0.56 ft</b>	
	$M_{R,bar}$	<b>-14.82 k-ft</b>	

Wearing Surface CG (MIN SLOPE)			
Dimensions	Area	$x_{cg}$	
$b_{A1} = 76.13 \text{ in}$ $h_{A1} = 11.00 \text{ in}$	$837.38 \text{ in}^2$	3.55 ft	
$b_{A2} = 76.13 \text{ in}$ $h_{A2} = 6.09 \text{ in}$	$-231.80 \text{ in}^2$	4.60 ft	
$b_{A3} = 76.13 \text{ in}$ $h_{A3} = 8.72 \text{ in}$	$-332.01 \text{ in}^2$	2.49 ft	
<b>TOTAL =</b>	<b><math>273.57 \text{ in}^2</math></b>	<b>3.93 ft</b>	
	$M_{R,ws}$	<b>41.85 k-ft</b>	

$\phi$	$M_R$	$\phi M_R$	$M_{OT}$	D/C	Check
0.5	215.24 k-ft	107.62 k-ft	56.7 k-ft	0.527	OK

Sensitive	<b>PARSONS</b>	MADE BY	DATE	CHK BY	DATE
		K. Garcia	9/15/2021	W. Metwally	11/17/2021
Job Number	WBS Number	TITLE		I-405 Renton to Bellevue	
649974	00531			Moment Slab Barrier	

#### [H] BEARING CAPACITY CHECK

$P_{slab}$ =	76.59 k	{slab load}
$P_{bar}$ =	26.47 k	{barrier load}
$P_{EV}$ =	28.67 k	{wearing surface load}

DC	STR III	STR IV	SER I	SER III	[AASHTO Table 3.4.1-1]
	1.25	1.50	1.00	1.00	
EV	1.35	1.35	1.00	1.00	
WS	1.00	0.00	1.00	0.00	

B =	6.72 ft
B/6 =	1.12 ft

Load	$x_{cg}$	$M_R$
Slab	-0.90 in <sup>2</sup>	<b>-69.1 k-ft</b>
Barrier	-3.92 ft	<b>-103.7 k-ft</b>

Conservatively ignore moment due to wearing surface.

P	STR III	STR IV	SER I	SER III	
P =	167.52 k	193.29 k	131.72 k	131.72 k	
M =	236.92 k-ft	259.22 k-ft	193.72 k-ft	172.81 k-ft	
e =	1.41 ft	1.34 ft	1.47 ft	1.31 ft	
$e < B/3 =$	OK	OK	OK	OK	
$q_{max} =$	1.44 ksf	1.60 ksf	1.16 ksf	1.07 ksf	
$w_{allow} =$	7.00 ksf	7.00 ksf	4.00 ksf	4.00 ksf	
Check =	OK	OK	OK	OK	

= M / P

[AASHTO 11.6.3.3]

[BDM 7.5.2.B.5]

#### [I] BARRIER MOMENT CAPACITY ABOUT VERTICAL AXIS, $M_w$

Determine factored flexural capacity

$f_c$ =	4.0 ksi
$f_y$ =	60.0 ksi
$\beta_1$ =	0.85
$\epsilon_{cu}$ =	0.003
$\epsilon_{slimit}$ =	0.005

$C_{ccover} =$  2.50 in.

b*	d*	$A_s$	$d_s$	c	$\epsilon_s$	Control	a	$\phi$	$\phi M_w$
63.00 in	14.94 in	1.24 in <sup>2</sup>	11.44 in	0.41 in.	0.0810	Tension	0.35 in.	0.9	11.97 k-ft

\*Assume average barrier dimensions

$A_s =$	$\Sigma N_{bi} A_{bi}$	
$d_{s,top} =$	$d - c_{ccover} - d_{bV} - 0.5d_{bLtop}$	
$c =$	$(A_s f_y) / (0.85 f_c \beta_1 b)$	[AASHTO-5.6.3.1.1-4]
$\epsilon_s =$	$(d_s - c) * \epsilon_{cu} / c$	[AASHTO 5.6.2.1]
$a =$	$\beta_1 c$	[AASHTO-5.6.2.2]
$\phi =$	$\max [ 0.75, \min [ 0.90, 0.75 + 50 * (\epsilon_s - 0.002) ] ]$	[AASHTO-5.6.2.2]
$\phi M_n =$	$\phi A_s f_y * (d_s - a / 2)$	[AASHTO-5.6.3.2.2-1]

#### [IJ] CALCULATE THE MOMENT CAPACITY OF THE BARRIER WALL ABOUT ITS BASE ( $M_c$ )

Determine factored flexural capacity

b	d	$A_s$	$d_s$	c	$\epsilon_s$	Control	a	$\phi$	$\phi M_c$
9.00 in	19.88 in	0.310 in. <sup>2</sup>	17.06 in	0.72 in.	0.0686	Tension	0.61 in.	0.9	31.17 k-ft

Sensitive	<b>PARSONS</b>	MADE BY	DATE	CHK BY	DATE
	K. Garcia	9/15/2021		W. Metwally	11/17/2021
Job Number	WBS Number	TITLE	I-405 Renton to Bellevue		
649974	00531		Moment Slab Barrier		

[K] CALCULATE THE YIELD LINE MECHANISM GEOMETRY AND UPPER BOUND RESISTANCE

L <sub>c,in</sub>	R <sub>w,in</sub>	L <sub>c,out</sub>	R <sub>w,out</sub>	R <sub>w</sub>	F <sub>T</sub>	D/C	Check
5.54 ft.	93.68 k	3.87 ft.	65.33 k	65.33 k	54.00 k	0.827	OK

$$L_{c,in} = L_t/2 + \sqrt{((L_t/2)^2 + (8H_{total}(M_{w1}+M_{w2})/M_c))}$$

[AASHTO A13.3.1-2]

$$R_{w,in} = (2/(2L_{c,in} \cdot L_t)) * [8(M_{w1}+M_{w2}) + M_c \cdot L_{c,in}^2 / H_{total}]$$

[AASHTO A13.3.1-1]

$$L_{c,out} = L_t/2 + \sqrt{((L_t/2)^2 + (H_{total}(M_{w1}+M_{w2})/M_c))}$$

[AASHTO A13.3.1-4]

$$R_{w,out} = (2/(2L_{c,out} \cdot L_t)) * [M_{w1}+M_{w2} + M_c \cdot L_{c,in}^2 / H_{total}]$$

[AASHTO A13.3.1-3]

$$R_w = \min(R_{w,in} + R_{w,out})$$

[L] CALCULATE THE MINIMUM LENGTH OF BARRIER TO PREVENT FAILURE AT THE BASE OF THE WALL

φM <sub>c</sub>	R <sub>w</sub>	L <sub>c,in</sub>	L <sub>c,out</sub>	L <sub>base</sub>	L <sub>min</sub>	L <sub>slab</sub>	D/C	Check
31.17 k-ft	65.33 k	5.54 ft.	3.87 ft.	7.73 ft.	7.73 ft.	40.00 ft	0.19333	OK

$$L_{base} = R_w H_{total} / M_c$$

$$L_{min} = \max(L_{c,in}, L_{c,out}, L_{base})$$

[M] INTERFACE SHEAR AT BOTTOM OF BARRIER

[AASHTO 5.7.4.]

For normal weight concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25"

c = 0.24 ksi	{cohesion factor}	K1 = 0.25		[AASHTO 5.7.4.4]
μ = 1	{friction factor}	K2 = 1.50 ksi		

b <sub>bar</sub> = 12.00 in	{1' strip width}	A <sub>vf,min</sub> = 0.20 in <sup>2</sup> /ft	{min. reqd shear reinf}	[AASHTO 5.7.4.2-1]
w <sub>bar</sub> = 19.88 in	{barrier length}	A <sub>vf,S3</sub> = 0.21 in <sup>2</sup> /ft		
A <sub>cv</sub> = 238.50 in <sup>2</sup>	{concrete interface area}	A <sub>vf,S4</sub> = 0.41 in <sup>2</sup> /ft		

$$A_{vf,prov} = 0.62 \text{ in}^2/\text{ft}$$

OK

$$P_c = 0.66 \text{ k} \quad \{ \text{permanent net compressive force normal to shear plane} = \text{barrier self weight} \}$$

$$V_{ni} = cA_{cv} + \mu(A_{vf}f_y + P_c) \quad [\text{AASHTO 5.7.43}]$$

V <sub>ni</sub> = 95.10 k	{nominal shear resistance}
V <sub>ui</sub> = 16.90 k	{shear demand = transverse collision forces}
D/C = 0.18	OK

[N] SLAB REINFORCEMENT FOR MOMENT RESISTANCE

**1. Determine factored flexural capacity**

f <sub>c</sub> = 4.0 ksi											
f <sub>y</sub> = 60.0 ksi											
β <sub>1</sub> = 0.85											
ε <sub>cu</sub> = 0.003	{concrete crushing strain}										
ε <sub>slimit</sub> = 0.005	{tension controlled steel strain minimum limit}										

$$C_{ccover} = 2.00 \text{ in.}$$

b	d	A <sub>s</sub>	d <sub>s</sub>	c	ε <sub>s</sub>	Control	a	ϕ	φM <sub>n</sub>
12.00 in	16.72 in	0.59 in <sup>2</sup>	14.35 in	1.01 in.	0.0394	Tension	0.86 in.	0.9	36.74 k-ft

M <sub>u</sub>	φM <sub>n</sub>	D/C	Check
19.21 k-ft	36.74 k-ft	0.523	OK

$$M_{u,slab} = R_w H / (L_c + 2H)$$

[AASHTO A13.4.2]

Sensitive	<b>PARSONS</b>	MADE BY	DATE	CHK BY	DATE
		K. Garcia	9/15/2021	W. Metwally	11/17/2021
Job Number	WBS Number	TITLE		I-405 Renton to Bellevue	
649974	00531			Moment Slab Barrier	

**2. Check minimum flexural reinforcement requirements.**

[AASHTO-5.6.3.3]

$$f_r = \boxed{0.48 \text{ ksi}} = 0.24\sqrt{(f_c)} \\ \gamma_1 = \boxed{1.6} \quad \{\text{non-segmental}\} \\ \gamma_2 = \boxed{1.0} \quad \{\text{no tendons}\} \\ \gamma_3 = \boxed{0.75} \quad \{\text{A706 reinforcement}\}$$

[AASHTO-5.4.2.6]  
[AASHTO-5.6.3.3]  
[AASHTO-5.6.3.3]  
[AASHTO-5.6.3.3]

$$f_{cpe} = \boxed{0.00 \text{ ksi}} \quad \{\text{no prestress force}\} \\ M_{dnc} = \boxed{0.00 \text{ k-ft}} \quad \{\text{conservatively set to zero}\}$$

S <sub>c</sub>	S <sub>nc</sub>	M <sub>cr</sub>
559.29 in <sup>3</sup>	559.29 in <sup>3</sup>	26.85 k-ft

$$S_c = bd^2 / 6$$

$$S_{nc} = S_c$$

$$M_{cr} = \gamma_3 * (S_c * (\gamma_1 f_r + \gamma_2 f_{cpe}) - M_{dnc} * (S_c / S_{nc} - 1))$$

[AASHTO-5.6.3.3-1]

1.33M <sub>u</sub>	M <sub>min</sub>	φM <sub>n</sub>	D / C	Check
25.55 k-ft	25.55 k-ft	36.74 k-ft/ft	<b>0.696</b>	<b>OK</b>

**[O] CHECK MINIMUM REINFORCEMENT FOR TEMPERATURE AND SHRINKAGE STEEL**

For bars, the area of reinforcement per foot, on each face and in each direction, shall satisfy:

$$1. A_s > (1.30bh)/(2(b+h)*f_y)$$

[AASHTO-5.10.6-1]

$$2. 0.11 < A_s < 0.60$$

[AASHTO-5.10.6-2]

$$3. s_{max} \leq 12"$$

[AASHTO-5.10.6]

Reinforcement							
b	h	A <sub>smin</sub>	A <sub>b</sub>	Check	s <sub>max</sub>	s <sub>b</sub>	Check
96.00 in	26.00 in	0.222 in <sup>2</sup>	0.330 in. <sup>2</sup>	<b>OK</b>	12.00 in	9.00 in	<b>OK</b>



## RETAINING WALL 09.90L - A & B

*2.2 —Moment Slab Barrier with Sign Structure Design*

<b>PARSONS</b>		MADE BY	DATE	CHK BY	DATE
		K. Garcia	11/30/2021	W. Metwally	11/30/2021
Job Number	WBS Number	TITLE	I-405 Renton to Bellevue	Moment Slab Barrier - Sign Bridge (9.90L)	
650512	00531				

#### [A] BASIS

- To check the Barrier Moment Slab Design supported by SE wall at the Sign Bridge location
- Autodesk Structural Bridge Design software is utilized to calculate the Design moment demand and capacity required for the slab bottom longitudinal bars.

#### [B] REFERENCES

Acronym	Source
AASHTO	AASHTO LRFD Bridge Design Specifications (8th Edition)
BDM	WSDOT Bridge Design Manual M23-50.18 (2018 Edition)
SUPP	AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (1st Ed.)

#### [C] CALCULATION LEGEND

-Standard cell format for User Input, Check Passed, No Good, and Output, respectively:

User Input	OK	NG	Output
------------	----	----	--------

-Curly brackets, { }, are used to enclose descriptions of inputs and calculations

-Square brackets, [ ], are used to enclose references

-All stations (##+###.###) have units 'm.' Other units are noted with their respective values.

#### [D] MATERIAL PROPERTIES

$\gamma_{c,E} =$	0.150 kcf	{reinforced concrete unit weight for capacity analysis}	[BDM Table 3.8-1]
$\gamma_c =$	0.145 kcf	{reinforced concrete unit weight}	[AASHTO-Table 3.5.1-1]
$\gamma_{pav} =$	0.140 kcf	{pavement unit weight}	[AASHTO-Table 3.5.1-1]
$\gamma_{soil} =$	0.120 kcf	{soil unit weight}	

#### Unconfined Concrete

$f_c$	$K_1$	$E_c$	$f_t$	$\varepsilon_{cu}$	$v_c$	$G_c$
4.0 ksi	1.00	4266.2 ksi	0.48 ksi	0.003	0.2	1777.6 ksi

$f_c =$  {concrete compressive strength }

[AASHTO-5.4.2.4-1]

$E_c =$  =  $120,000K_1\gamma_c^2 f_c^{0.33}$

[AASHTO-5.4.2.6]

$f_t =$  =  $0.24\sqrt{f_c}$  {concrete tension capacity for this analysis}

[AASHTO-5.6.2.1]

$\varepsilon_{cu} =$  {maximum unconfined concrete strain}

[AASHTO 5.4.2.5]

$v_c =$  {poisson's ratio for concrete}

[AASHTO 5.4.2.5]

$G_c =$   $E_c / (2*(1+v_c))$  {concrete shear modulus}

#### ASTM A706 Grade 60 Reinforcing Steel

$f_y$	$f_u$	$E_s$	$\varepsilon_y$
60.0 ksi	80.0 ksi	29000 ksi	0.0021

$f_y =$  {minimum yield strength}

[ASTM A706-16 Table A1.2]

$f_u =$  {minimum tensile strength}

[ASTM A706-16 Table A1.2]

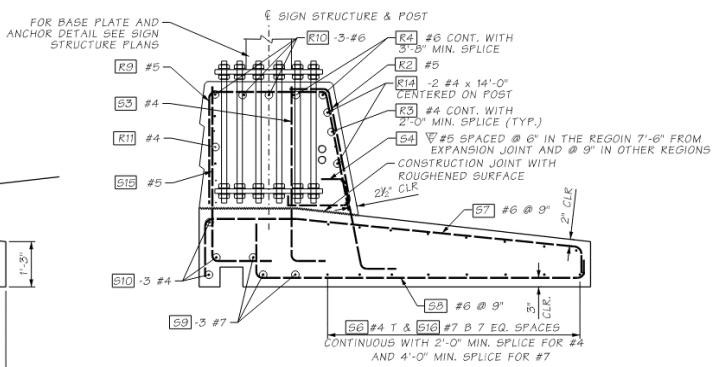
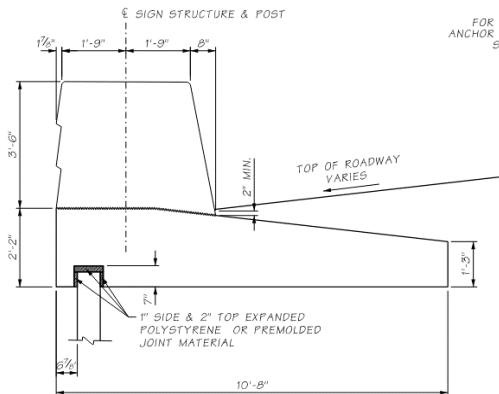
$E_s =$  {steel modulus of elasticity}

[AASHTO 5.4.3.2]

$\varepsilon_y =$  {nominal yield strain}

PARSONS		MADE BY	DATE	CHK BY	DATE		
K. Garcia		11/30/2021		W. Metwally			
Job Number	WBS Number	TITLE	I-405 Renton to Bellevue				
650512	00531		Moment Slab Barrier - Sign Bridge (9.90L)				

## [E] SLAB GEOMETRY AND REINFORCEMENT



## 1. Barrier and Slab Geometry

Slab:

$t_{\text{slab,min}}$	15.00 in	{min. slab thickness}
$t_{\text{slab,max}}$	26.00 in	{max. slab thickness}
$\text{cover}$	2.00 in	{concrete cover}
$b_{\text{side,gap}}$	1.00 in	{side gap width @ notch}
$b_{\text{top,gap}}$	2.00 in	{top gap height @ notch}
$b_{\text{wall}}$	7.50 in	{wall thickness}
$b_{\text{notch}}$	9.50 in	{total notch width}
$t_{\text{notch}}$	7.00 in	{total notch height}
$b_{\text{OHslab}}$	5.88 in	{slab edge to wall edge}
$b_{\text{slab,slope}}$	8.00 ft	{slab width sloped}
$b_{\text{slab,tot}}$	10.67 ft	{total slab width}
$L_{\text{slab,min}}$	34.00 ft	{minimum slab length}
$L_{\text{slab,max}}$	120.00 ft	{maximum slab length}
$A_{\text{slab,wide}}$	2733.50 in <sup>2</sup>	{slab cross-sectional area}
$A_{\text{slab,typ}}$	1901.50 in <sup>2</sup>	{slab cross-sectional area}
$V_{\text{slab}}$	576.08 ft <sup>3</sup>	{slab volume}

Barrier:

$h_{\text{barrier}} =$	42.00 in	{barrier height total}
$b_{\text{bar,top}} =$	42.00 in	{barrier width @ top}
$b_{\text{bar,bot}} =$	19.88 in	{barrier width @ bottom}
$b_{\text{finish1}} =$	1.88 in	{barrier finish width}
$b_{\text{finish2}} =$	1.50 in	{barrier finish width}
$h_{\text{finish}} =$	1.00 in	{barrier finish height}
$h_{\text{seg}} =$	14.00 in	{barrier finish segment height}
$A_{\text{bar,wide}} =$	2001.13 in <sup>2</sup>	{barrier cross-sectional area @ widened section}
$A_{\text{bar,typ}} =$	657.13 in <sup>2</sup>	{barrier cross-sectional area typical section}
$V_{\text{bar}} =$	360.49 ft <sup>3</sup>	{barrier volume}

$$A_{ftg} = \boxed{287.10 \text{ ft}^2}$$

### **Check Max and Min Geometry:**

$b_{\text{slab,min,req}}$	4.00 ft	OK	required minimum slab width
$t_{\text{slab,avg,req}}$	0.83 ft	OK	required average slab depth

[BDM 10.3.2-3]

[BDM 10.3.2-3]

#### **Check Torsional Rigidity for Max Slab Length:**

2a=	128.00 in	{total width of slab}
2b=	20.50 in	{average depth of slab}
a=	64.00 in	
b=	10.25 in	
J=	330492.70 in4	
J60=	13900.17 in4	
I(=)	120.00 in <sup>3</sup>	OK

$$J = a \cdot b^3 \left[ \frac{16}{3} - 3.36 \left( \frac{b}{a} \right) \left( 1 - \frac{b^4}{12a^4} \right) \right]$$

L(max)= 120.00 ft OK {required maximum slab length}

[BDM 10.3.2-3]

PARSONS		MADE BY		DATE		CHK BY	DATE		
		K. Garcia		11/30/2021		W. Metwally	11/30/2021		
Job Number	WBS Number	TITLE	I-405 Renton to Bellevue						
650512	00531		Moment Slab Barrier - Sign Bridge (9.90L)						

## 2. Barrier Reinforcement

Bar Mark	Bar Size	d <sub>b,L</sub>	A <sub>b,L</sub>	N <sub>b,L</sub>	s <sub>b</sub>	
R3+R14	#4	0.50 in	0.20 in <sup>2</sup>	6	6.00 in	{continuous}
R11+R14	#4	0.50 in	0.20 in <sup>2</sup>	6	6.00 in	{continuous}
R4+R10	#6	0.75 in	0.44 in <sup>2</sup>	5	8.00 in	{continuous}
R9	#5	0.63 in	0.31 in <sup>2</sup>	4	18.00 in	

Bar Mark	Bar Size	d <sub>b,V</sub>	A <sub>b,V</sub>	s <sub>b,V</sub>	N <sub>b,L</sub>
R2	#5	0.63 in	0.31 in <sup>2</sup>	9.00 in	-
S3	#4	0.50 in	0.20 in <sup>2</sup>	18.00 in	2
S4	#5	0.63 in	0.31 in <sup>2</sup>	9.00 in	-

## 3. Slab Reinforcement

Bar Mark	Bar Size	d <sub>b,L</sub>	A <sub>b,L</sub>	N <sub>b,L</sub>	s <sub>b,L</sub>	
S10	#4	0.50 in	0.20 in <sup>2</sup>	3	-	{continuous}
S6	#4	0.50 in	0.20 in <sup>2</sup>	8	10.41 in	{continuous}
S16+S9	#7	0.88 in	0.60 in <sup>2</sup>	10	11.81 in	{continuous}

Bar Mark	Bar Size	d <sub>b,V</sub>	A <sub>b,V</sub>	s <sub>b,V</sub>
S7	#6	0.75 in	0.44 in <sup>2</sup>	9.00 in
S8	#6	0.75 in	0.44 in <sup>2</sup>	9.00 in

## 4. Check Maximum Spacing of Reinforcement

[AASHTO 5.10.3.2]

s <sub>b,L</sub>	s <sub>max</sub>	Check	s <sub>b,V</sub>	s <sub>max</sub>	Check
10.41 in.	18.00 in.	OK	9.00 in.	18.00 in.	OK
11.81 in.	18.00 in.	OK	9.00 in.	18.00 in.	OK

## [F] LOADING

### Barrier Slab Dead Loads

Slab Weight	Barrier
W <sub>slab</sub>	W <sub>bar</sub>
83.53 k	52.27 k

### Sign Bridge Dead Loads

[BDM Table 10.1.4-3]

Table 10.1.4-3 Sign Structure Material Quantities											
ASTM A572 GR. 50 or ASTM 588	Cantilever		Sign Bridge								
	20' < to 30'	Balanced	60' < 75'	75' to 90'	90' to 105'	105' to 120'	120' to 135'	135' to 150'	150' to 180'		
Post (plf)	132	132	132	176	176	204	204	215	215	267	
Base PL (lbs./ea)	537	806	800	672	735	735	888	888	978	978	1029
Beam, near Post (plf)	152	152	152	202	202	228	228	240	240	257	
Span Beam (plf)	152	152	152	202	202	228	228	240	240	257	
Corner Stiff. (lbs./ea set)	209	209	115	218	272	272	354	354	376	425	
Splice PI #1 (lbs/pair)	592	706	706	578	650	650	826	826	1116	1116	1295
Splice PI #2 (lbs/pair)	--	--	--	--	730	730	1002	1002	1116	1116	1295
Splice PI #3 (lbs/pair)	--	--	--	--	--	--	--	--	--	--	1295
Brackets (lbs./ea)	60	60	60	60	65	65	69	69	70	70	70
6" Hand Hole (lbs./ea)	18	18	18	18	18	18	18	18	18	18	18
6" x 11" Hand Hole (lbs./ea)	30	30	30	30	30	30	30	30	30	30	30
Anchor Bolt PL (lbs./ea)	175	175	175	175	185	185	311	311	326	326	326
Cover Plates (lbs./ea)	65	65	65	--	--	--	--	--	--	--	--

Element	Quantity	Length	Weight	W <sub>tot</sub>
Post 1	1	25.1 ft	204.0 plf	5117.0 lb
Beam A	1	31.7 ft	228.0 plf	7229.5 lb
Span Beam	1	20.0 ft	228.0 plf	4560.0 lb
Base PL	1	-	888.0 lb	888.0 lb
Corner Stiff	2	-	354.0 lb	708.0 lb
Splice #1	1	-	826.0 lb	826.0 lb
Splice #2	1	-	1002.0 lb	1002.0 lb
Brackets	4	-	69.0 lb	276.0 lb
Anchor PL	1	-	311.0 lb	311.0 lb
Self Weight =				20.9 k

PARSONS		MADE BY	DATE	CHK BY	DATE
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### Sign Dead Loads

$$\gamma_{\text{sign}} = \boxed{3.25 \text{ psf}} \quad \{\text{sign weight}\} \quad [\text{BDM 10.1.1}]$$

Sign	L	H	W <sub>sign</sub>
36-7	29.00 ft	18.00 ft	1696.50 lb
36-15A	20.00 ft	14.50 ft	942.50 lb
36-15B	6.00 ft	2.50 ft	48.75 lb
36-27	4.00 ft	5.00 ft	65.00 lb
36-28	5.00 ft	3.50 ft	56.88 lb
<b>0.5*Total Sign Weight =</b>			<b>1.47 k</b>

### Soil Weight Above Slab

d <sub>min</sub> =	0.19 ft	{depth of roadway above slab @ face of barrier}
slope =	-8.00%	{roadway slope}
t <sub>slab,bar</sub> =	1.98 ft	{moment slab depth at face of barrier}
d <sub>max</sub> =	0.41 ft	{depth of roadway above slab @ edge of slab}
A <sub>ws</sub> =	1.90 ft <sup>2</sup>	{area of wearing surface}
V <sub>ws</sub> =	64.59 ft <sup>3</sup>	{volume of wearing surface above slab}
w <sub>ws</sub> =	0.14 kcf	{wearing surface unit weight}
W <sub>ws</sub> =	<b>9.04 k</b>	

[BDM Table 3.8-1]

Unfactored Axial Loads	
Element	P <sub>u</sub> (axial)
Barrier Slab	135.80 k
Sign Bridge	20.92 k
Signs	1.47 k
Soil Weight	9.04 k

### Sign Bridge Wind Loads (Longitudinal)

$$P_z = 0.00256 K_z G V^2 I_r C_d \quad [\text{SUPP 3.8.3-1}]$$

V =	115.0 mph	{3-sec gust wind speed}	[SUPP Table 3.8.3-1]
WEC =	C	{Wind Exposure Category}	
I <sub>r</sub> =	1.00	{importance factor}	[SUPP 3.8.3-1]
Z =	38.00 ft	{height to top of sign}	[SUPP C3.8.4-1]
$\alpha$ =	9.50 ft		
z <sub>g</sub> =	900.00 ft		
K <sub>z</sub> =	1.03	{height factor}	[SUPP 3.8.6]
G =	1.14	{gust factor}	[SUPP Table 3.8.7-1]

Sign	L	W	L/W	C <sub>d</sub>	P <sub>z</sub>	A <sub>tot</sub>	F	Z <sub>ws</sub>	M <sub>ws</sub>
36-7	29.00 ft	18.00 ft	1.61	1.16	46.10 psf	522.00 ft <sup>2</sup>	24.06 k	31.00 ft	746.0 k-ft
36-15A	20.00 ft	14.50 ft	1.38	1.15	45.46 psf	290.00 ft <sup>2</sup>	13.18 k	31.00 ft	408.7 k-ft
36-15B	6.00 ft	2.50 ft	2.40	1.19	47.23 psf	15.00 ft <sup>2</sup>	0.71 k	39.50 ft	28.0 k-ft
36-27	4.00 ft	5.00 ft	1.25	1.14	45.10 psf	20.00 ft <sup>2</sup>	0.90 k	11.67 ft	10.5 k-ft
36-28	5.00 ft	3.50 ft	1.43	1.15	45.59 psf	17.50 ft <sup>2</sup>	0.80 k	15.92 ft	12.7 k-ft
Post 1	25.08 ft	1.50 ft	-	1.70	67.40 psf	37.63 ft <sup>2</sup>	2.54 k	18.46 ft	46.8 k-ft
Beam A	31.71 ft	2.00 ft	-	1.70	67.40 psf	63.42 ft <sup>2</sup>	4.27 k	31.00 ft	132.5 k-ft
Span Beam	20.00 ft	2.00 ft	-	1.70	67.40 psf	40.00 ft <sup>2</sup>	2.70 k	31.00 ft	83.6 k-ft

<b>PARSONS</b>		MADE BY	DATE	CHK BY	DATE
		K. Garcia	11/30/2021	W. Metwally	11/30/2021
Job Number	WBS Number	TITLE	I-405 Renton to Bellevue	Moment Slab Barrier - Sign Bridge (9.90L)	
650512	00531				

Sign	Z <sub>WS@base</sub>	M <sub>ws</sub> at base
36-7	25.08 ft	603.62 k-ft
36-15A	25.08 ft	330.66 k-ft
36-15B	33.58 ft	23.79 k-ft
36-27	9.50 ft	8.57 k-ft
36-28	13.75 ft	10.97 k-ft
Post 1	12.54 ft	31.80 k-ft
Beam A	25.08 ft	107.21 k-ft
Span Beam	25.08 ft	67.62 k-ft

P<sub>zmax</sub> = {design wind pressure on sign}

A<sub>tot</sub> = {total sign area}

P<sub>wind</sub> = {total wind load on sign}

Assume sign wind loads distribute equally to each post

$$F_{WS,L} = \boxed{30.18 \text{ k}} \quad \{\text{longitudinal wind force}\}$$

$$M_{WS,L} = \boxed{877.43 \text{ k-ft}} \quad \{\text{longitudinal wind moment}\}$$

Base plate demands

$$F_{WS,L} = \boxed{30.18 \text{ k}} \quad \{\text{longitudinal wind force}\}$$

$$M_{WS,L} = \boxed{705.22 \text{ k-ft}} \quad \{\text{longitudinal wind moment}\}$$

$$\text{Axial Force} = \boxed{22.38 \text{ k-ft}}$$

#### Barrier Wind Loads (Transverse)

-perpendicular to interior barrier face controls transverse loading

Wind Exposure Category C

[BDM 3.11.1]

$$P_z = 2.56 \times 10^{-6} V^2 K_z G C_D$$

$$Z = \boxed{33.00 \text{ ft}} \quad \{\text{height}\}$$

[AASHTO Table 3.8.1.1.2-1]

$$V = \boxed{115.00 \text{ mph}} \quad \{\text{gust wind speed}\}$$

[AASTHO 3.8.1.2.1-2]

$$K_z = \boxed{1.00} \quad \{\text{pressure exposure and elevation coefficient}\}$$

[AASHTO Table 3.8.1.2.1-1]

$$G = \boxed{1.0} \quad \{\text{gust effect factor}\}$$

[AASHTO Table 3.8.1.2.1-2]

$$C_D = \boxed{1.2} \quad \{\text{drag coefficient}\}$$

[AASHTO 3.8.1.2.1-1]

$$P_z = \boxed{0.041 \text{ ksf}}$$

STRI	
w <sub>T,b</sub>	= 0.142 klf {barrier total transverse uniform wind line load}
z <sub>eg,WS,b</sub>	= 1.75 ft {centroid of barrier wind load above point of rotation}
F <sub>WS,T</sub>	= 0.142 k {barrier total transverse wind load}
M <sub>WS,T</sub>	= 0.81 k-ft/ft {transverse uniform WS moment about POR}

[AASHTO 3.8.1.2.1-1]

Wind Cases		
Case	Longitudinal	Transverse
1	1.00	0.00
2	0.00	1.00
3	0.75	0.75

{fully longitudinal}  
{fully transverse}  
{both}

Wind Demands				
Case	F <sub>long</sub>	F <sub>trans</sub>	M <sub>long</sub>	M <sub>trans</sub>
1	30.18 k	0.00 k	877.43 k	0.00 k
2	0.00 k	4.84 k	0.00 k	27.44 k
3	22.64 k	3.63 k	658.08 k	20.58 k

<b>PARSONS</b>		MADE BY	DATE	CHK BY	DATE
K. Garcia		11/30/2021		W. Metwally	
Job Number	WBS Number	TITLE	I-405 Renton to Bellevue		
650512	00531		Moment Slab Barrier - Sign Bridge (9.90L)		

[G] GLOBAL STABILITY CHECK

**Check Sliding Stability**

$\gamma_{soil}$	$\delta_{soil}$	$\gamma$	$\phi_{ext}$
0.120 kcf	32.0 °	1	0.8

$$DC = \boxed{158.19 \text{ k}} \quad \{\text{dead load axial load}\}$$

$$EV = \boxed{9.04 \text{ k}} \quad \{\text{earth axial load}\}$$

Limit	$P_u$	$R_{slide}$	$F_{slide}$			D/C
			Case 1	Case 2	Case 3	
EEI Min	150.51 k	<b>94.05 k</b>	30.18 k	4.84 k	22.93 k	<b>0.321</b>
EEI Max	183.95 k	<b>114.95 k</b>				

$\gamma_{soil}$  = {unit weight, granular base material}

$\delta_{soil}$  = {Friction Angle, granular base material}

$R_{slide} = W_{total} * \tan(\delta_{soil})$  {Frictional resistance to sliding}

$F_{slide} = \sqrt{F_t^2 + F_l^2}$  {sliding force on Moment Slab}

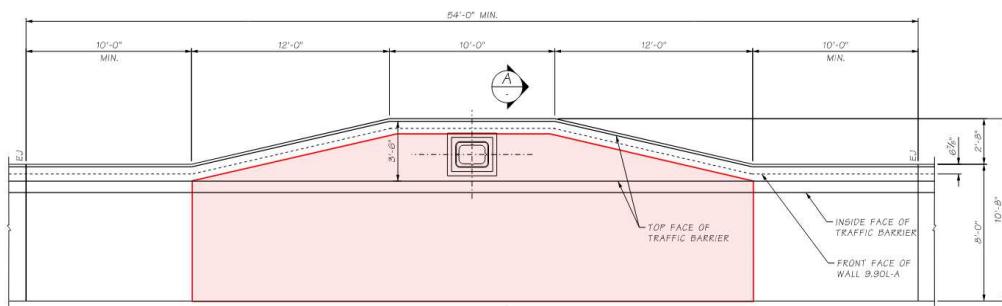
$\gamma$  = {load factor}

[BDM 10.3.2-B-4]

**Check Overturning Stability**

Uplift is not permitted in uniaxial bending. The uplift area shall not exceed 25% of the total bearing area,

[SUPP 13.7.1]



TYPICAL PLAN - MOMENT SLAB W/ SIGN STRUCTURE OR MAST ARM

	STR I		EE I	
	Max	Min	Max	Min
DC =	1.25	1.25	1.10	0.90
EV =	1.25	1.25	1.10	0.90
WS =	0.00	0.00	1.00	1.00

<b>PARSONS</b>		MADE BY	DATE	CHK BY	DATE
		K. Garcia	11/30/2021	W. Metwally	11/30/2021
Job Number	WBS Number	TITLE	I-405 Renton to Bellevue	Moment Slab Barrier - Sign Bridge (9.90L)	
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**CASE 1: Slab is longitudinally symmetrical - only the Wind on Sign Bridge moments are overturning in the longitudinal direction.**

Case 1 Longitudinal				
Limit	P	M <sub>o</sub>	e <sub>l</sub>	e <sub>limit</sub>
STRI Min	209.04 k	0.00 k-ft	0.00 ft	11.33 ft
STRI Max	209.04 k	0.00 k-ft	0.00 ft	11.33 ft
EE I Min	150.51 k	877.43 k-ft	5.83 ft	11.33 ft
EE I Max	183.95 k	877.43 k-ft	4.77 ft	11.33 ft

**CASE 2: Slab is transversely asymmetrical - Slab, Barrier and Sign Bridge weights plus Barrier wind load induces moment in transverse direction**

$$B = \boxed{8.44 \text{ ft}}$$

Element	X <sub>cg</sub>	P <sub>DC</sub>	M <sub>DC</sub>
Wide Barrier	4.39 k	20.15 k	88.52 k-ft
Taper Barrier	3.70 k	32.12 k	118.89 k-ft
Wide Footing	1.47 k	27.52 k	40.50 k-ft
Taper Footing	0.73 k	56.01 k	41.00 k-ft
Wear. Surf.	-1.48 k	9.04 k	-13.42 k-ft
Sign Bridge	4.49 k	20.92 k	93.95 k-ft

\*conservatively ignore any negative moments

Case 2 Transverse				
Limit	P	M <sub>o</sub>	e <sub>t</sub>	e <sub>limit</sub>
STRI Min	209.04 k	461.80 k-ft	2.21 ft	2.81 k-ft
STRI Max	209.04 k	461.80 k-ft	2.21 ft	2.81 k-ft
EE I Min	150.51 k	359.93 k-ft	2.39 ft	2.81 k-ft
EE I Max	183.95 k	433.82 k-ft	2.36 ft	2.81 k-ft

**CASE 3: Assume bidirectional moments - use vector sum of longitudinal and transverse:**

Case 3 Combined						
Limit	P	M <sub>l</sub>	M <sub>t</sub>	M <sub>o</sub>	e <sub>c</sub>	e <sub>limit</sub>
STRI Min	209.04 k	0.00 k-ft	461.80 k-ft	461.80 k-ft	2.21 ft	11.68 ft
STRI Max	209.04 k	0.00 k-ft	461.80 k-ft	461.80 k-ft	2.21 ft	11.68 ft
EE I Min	150.51 k	658.08 k-ft	353.07 k-ft	746.81 k-ft	4.96 ft	11.68 ft
EE I Max	183.95 k	658.08 k-ft	426.96 k-ft	784.45 k-ft	4.26 ft	11.68 ft

Limit	$\sigma_{min}$	$\sigma_1$	$\sigma_2$	$x_1$	$x_2$	$A_{tens}$	$A_{t,max}$
STRI Min	-0.35 ksf	1.80 ksf	-0.35 ksf	1.36 ft	34.00 ft	46.25 ft <sup>2</sup>	71.78 ft <sup>2</sup>
STRI Max	-0.35 ksf	1.80 ksf	-0.35 ksf	1.36 ft	34.00 ft	46.25 ft <sup>2</sup>	71.78 ft <sup>2</sup>
EE I Min	-0.74 ksf	0.90 ksf	0.15 ksf	3.82 ft	28.32 ft	54.10 ft <sup>2</sup>	71.78 ft <sup>2</sup>
EE I Max	-0.80 ksf	1.19 ksf	0.09 ksf	3.39 ft	30.43 ft	51.65 ft <sup>2</sup>	71.78 ft <sup>2</sup>

$$\sigma_{min} = P_u/A - M_{o,n}c_n/l_n - M_{o,t}c_t/l_t$$

$$\sigma_1 = P_u/A - M_{o,n}c_n/l_n + M_{o,t}c_t/l_t$$

$$\sigma_2 = P_u/A + M_{o,n}c_n/l_n - M_{o,t}c_t/l_t$$

<b>PARSONS</b>		MADE BY	DATE	CHK BY	DATE
		K. Garcia	11/30/2021	W. Metwally	11/30/2021
Job Number 650512	WBS Number 00531	TITLE	I-405 Renton to Bellevue		
			Moment Slab Barrier - Sign Bridge (9.90L)		

#### [H] BEARING CAPACITY CHECK

##### CASE 1:

Limit	e	$\sigma_v$	w <sub>allow</sub>	D/C
STRI Min	0.00 ft	0.73 ksf	7.00 ksf	0.10
STRI Max	0.00 ft	0.73 ksf		0.10
EE I Min	5.83 ft	0.80 ksf	7.00 ksf	0.11
EE I Max	4.77 ft	0.89 ksf		0.13

##### CASE 2:

Limit	e	$\sigma_v$	w <sub>allow</sub>	D/C
STRI Min	2.21 ft	1.53 ksf	7.00 ksf	0.22
STRI Max	2.21 ft	1.53 ksf		0.22
EE I Min	2.39 ft	1.21 ksf	7.00 ksf	0.17
EE I Max	2.36 ft	1.45 ksf		0.21

##### CASE 3:

Limit	e <sub>l</sub>	$\sigma_{vl}$	e <sub>t</sub>	$\sigma_{vt}$	$\sigma_{tot}$	w <sub>allow</sub>	D/C
STRI Min	0.00 ft	0.73 ksf	2.21 ft	1.53 ksf	1.53 ksf	7.00 ksf	0.22
STRI Max	0.00 ft	0.73 ksf		1.53 ksf	1.53 ksf		0.22
EE I Min	4.37 ft	0.71 ksf	2.35 ft	1.18 ksf	1.36 ksf	7.00 ksf	0.19
EE I Max	3.58 ft	0.81 ksf		1.42 ksf	1.59 ksf		0.23

#### [I] MOMENT & SHEAR DEMANDS FOR REINFORCEMENT DESIGN

l =	34.00 ft	{foundation length}
b =	6.34 ft	{foundation width}
c =	0.58 ft	{edge of slab to CL Sign Bridge}
A <sub>slab</sub> =	10.24 ft <sup>2</sup>	{slab area at the face of barrier}
W <sub>slab</sub> =	1.48 k	{slab weight at the face of barrier}
X <sub>slab-barrier</sub> =	2.93 ft	
X <sub>pav-barrier</sub> =	3.57 ft	
X <sub>slab1-sign</sub> =	10.27 ft	

Limit	$\sigma_{v,trans}$	*+M <sub>trans</sub> @ barrier face	-M <sub>trans</sub> @ barrier face	*+V <sub>trans</sub> @ barrier face	+M <sub>DL+EVlong</sub> @ CL sign br.	+M <sub>long</sub> @ CL sign br.	*+V <sub>long</sub> @ barrier face
STRI Min	1.53 ksf	24.10 k-ft	see TYP Moment Slab calcs	7.50 k	688.01 k-ft	1175.43 k-ft	128.70 k
STRI Max	1.53 ksf	24.10 k-ft		7.50 k	688.01 k-ft	1175.43 k-ft	128.70 k
EE I Min	1.36 ksf	22.62 k-ft		7.06 k	495.36 k-ft	1165.56 k-ft	130.22 k
EE I Max	1.59 ksf	26.24 k-ft		8.19 k	605.44 k-ft	1339.23 k-ft	149.12 k

\*values are per foot length

<b>PARSONS</b>		MADE BY	DATE	CHK BY	DATE
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650512	00531				

## [N] SLAB REINFORCEMENT FOR MOMENT RESISTANCE

### 1. Determine factored flexural capacity (longitudinal)

Moment demand = wind on sign bridge

$f_c =$	4.0 ksi	{concrete strength}	
$f_y =$	60.0 ksi	{reinforcing steel strength}	
$\beta_1 =$	0.85		[AASHTO-5.6.2.2]
$\epsilon_{cu} =$	0.003	{concrete crushing strain}	
$\epsilon_{slimit} =$	0.005	{tension controlled steel strain minimum limit}	[AASHTO-5.6.2.1]

$C_{cbot} =$	3.00 in.	{clear cover bottom}
$C_{ctop} =$	2.00 in.	{clear cover top}
$b =$	12.00 in	{unit strip width}
$d =$	20.50 in	{average slab depth}
$d =$	16.72 in	{slab depth at face of barrier}

Reinf.	$A_s$	$d_s$	$c$	$\epsilon_s$	Control	$a$	$\phi$	$\phi M_n$
Bot Long	-	-	-	From Autodesk Bridge Design software		0.9	1944.86 k-ft	
Bot Trans	0.59 in <sup>2</sup>	13.35 in	1.01 in.	0.0365	Tension	0.86 in.	0.9	34.10 k-ft
Top Trans	0.59 in <sup>2</sup>	14.35 in	1.01 in.	0.0394	Tension	0.86 in.	0.9	36.74 k-ft

Reinf.	$\phi M_n$	$M_u$	D/C	Check
Bot Long	1944.86 k-ft	1339.23 k-ft	<b>0.689</b>	OK
Bot Trans	34.10 k-ft	26.24 k-ft	<b>0.769</b>	OK
Top Trans	36.74 k-ft	19.21 k-ft	<b>0.523</b>	OK

\*Assume average barrier dimensions

$A_s =$	$\sum N_{bi} A_{bi}$		
$d_s =$	$d - c_{ccover} - d_{bV} - 0.5d_{bLtop}$		
$c =$	$(A_s f_y) / (0.85 f_c \beta_1 b)$		[AASHTO-5.6.3.1.1-4]
$\epsilon_s =$	$(d_s - c) * \epsilon_{cu} / c$		[AASHTO 5.6.2.1]
$a =$	$\beta_1 c$		[AASHTO-5.6.2.2]
$\phi =$	$\max [ 0.75, \min [ 0.90, 0.75 + 50 * (\epsilon_s - 0.002) ] ]$		[AASHTO-5.6.2.2]
$\phi M_n =$	$\phi A_s f_y * (d_s - a / 2)$		[AASHTO-5.6.3.2.2-1]

### 2. Check minimum flexural reinforcement requirements.

[AASHTO-5.6.3.3]

$f_r =$	0.48 ksi	= $0.24\sqrt{f_c}$		[AASHTO-5.4.2.6]
$\gamma_1 =$	1.6	{non-segmental}		[AASHTO-5.6.3.3]
$\gamma_2 =$	1.0	{no tendons}		[AASHTO-5.6.3.3]
$\gamma_3 =$	0.75	{A706 reinforcement}		[AASHTO-5.6.3.3]
$f_{cpe} =$	0.00 ksi	{no prestress force}		
$M_{dnc} =$	0.00 k-ft	{conservatively set to zero}		

$S_c$	$S_{nc}$	$M_{cr}$	
-	-	3394.58 k-ft	From Autodesk Bridge design software
559.29 in <sup>3</sup>	559.29 in <sup>3</sup>	26.85 k-ft	

$$S_c = bd^2 / 6$$

$$S_{nc} = S_c$$

$$M_{cr} = \gamma_3 * (S_c * (\gamma_1 f_r + \gamma_2 f_{cpe}) - M_{dnc} * (S_c / S_{nc} - 1))$$

[AASHTO-5.6.3.3-1]

Reinf.	$1.33M_u$	$M_{rmin}$	$\phi M_n$	D / C	Check
Bot Long	1781.18 k-ft	1781.18 k-ft	1944.86 k-ft/ft	<b>0.916</b>	OK
Bot Trans	34.89 k-ft	26.85 k-ft	34.10 k-ft/ft	<b>0.787</b>	OK
Top Trans	25.55 k-ft	25.55 k-ft	36.74 k-ft/ft	<b>0.696</b>	OK

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650512	00531				

#### [O] SHEAR CAPACITY (STRENGTH)

Shear resistance shall be taken as the lesser of the following:

$$V_{n1} = V_c + V_s$$

$$V_{n2} = 0.25 f_c b_v d_v$$

where:

$$V_c = 0.0316 \beta \lambda \sqrt{f_c} b_v d_v$$

$$V_s = A_v f_y d_v / s$$

$\beta =$	2.00	
$b_v =$	12.00 in	{effective web width}
$d_v =$	17.08 in	{effective shear depth; MAX( 0.90d_e or 0.72h)}
$V_u =$	8.19 kip	{shear demand}
$V_c =$	25.91 kip	{concrete shear resistance}
$V_{n1} =$	25.91 kip	[AASHTO-5.7.3.3-3]
$V_{n2} =$	204.96 kip	[AASHTO-5.7.3.3-1]
$\phi =$	0.90	[AASHTO-5.7.3.3-2]
$\phi_v V_n =$	23.32 kip	[AASHTO-5.7.3.3]
D/C =	0.351	

#### [P] CHECK MINIMUM REINFORCEMENT FOR TEMPERATURE AND SHRINKAGE STEEL

For bars, the area of reinforcement per foot, on each face and in each direction, shall satisfy:

1.  $A_s > (1.30bh)/(2(b+h)*f_y)$  [AASHTO-5.10.6-1]
2.  $0.11 < A_s < 0.60$  [AASHTO-5.10.6-2]
3.  $s_{max} \leq 12"$  [AASHTO-5.10.6]

Reinf.	Reinforcement							
	b	h	$A_{smin}$	$A_b$	Check	$s_{max}$	$s_b$	Check
R3+R14	46.94 in	42.00 in	0.240 in <sup>2</sup>	0.343 in. <sup>2</sup>	OK	12.00 in	6.00 in	OK
R11+R14	46.94 in	42.00 in	0.240 in <sup>2</sup>	0.343 in. <sup>2</sup>	OK	12.00 in	6.00 in	OK
R10+R4	46.94 in	42.00 in	0.240 in <sup>2</sup>	0.562 in. <sup>2</sup>	OK	12.00 in	8.00 in	OK



Job: I-405 Renton to Bellevue - Moment Slab Barrier - Sign Bridge (9.90L)

Job No.: 650512  
Calc. By: WM

Section: Moment Slab  
Longitudinal Moment

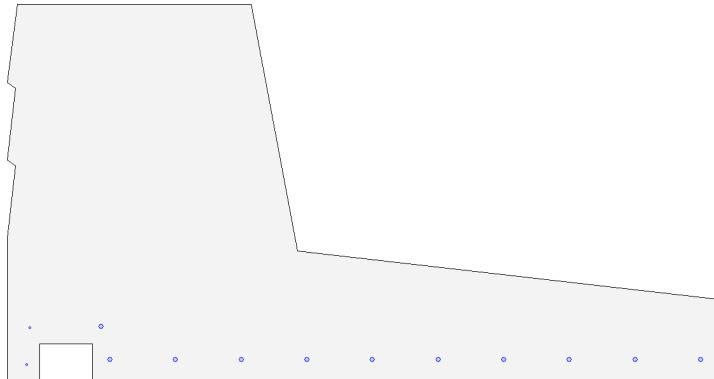
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## Data Report

### Data Report Links

[Materials](#)  
[Design Section](#)



## MATERIALS

### MP1: C4 Es 4.3E3

#### Type: Concrete - Parabola-Rectangle

Compressive strength  $f'_c$ : 3.9999999 KSI

Strength LS stress/strain curve parameters:  
slope at start of parabola: 4718.5148 KSI  
strain at end of parabola: 0.0014433  
stress at end of parabola: 3.3999931 KSI  
maximum strain: 0.003

Elastic modulus -  $E_c$ : 4266.2 KSI

Elastic modulus - long term: 2133.1 KSI

Shear modulus: 1777.5834 KSI

Poisson's ratio: 0.2

Compressive Stress limit factor:  $0.6000000 \times f'_c$

Modulus of Rupture: -0.48 KSI

Coefficient of thermal expansion: 0.0000108 /°C

Correction factor for source of

aggregate K1: 1.0

Maximum aggregate size: 1.0 IN

Density: 0.1450000 KIP/FT<sup>3</sup>

Density Modification Factor,  $\lambda$ : 1.0



Job: I-405 Renton to Bellevue - Moment Slab Barrier - Sign Bridge (9.90L)

Job No.: 650512  
Calc. By: WM

Section: Moment Slab  
Longitudinal Moment

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**MP2: Grade 60 E 2.9E4**  
**Type: Reinforcing Steel**

Yield strength  $f_y$ : 60.000002 KSI

Strength LS stress/strain curve parameters:

	strain	stress
		KSI
Tension		
full yield:	-0.004069	-6.0E1
start yield:	-0.001655	-4.8E1
Compression		
start yield:	0.0016552	48.000002
full yield:	0.002	49.716506

Elastic modulus  $E_s$ : 28999.999 KSI

Shear modulus: 11153.846 KSI

Max. stress limitation factor: 0.6000000 x  $f_y$



Job: I-405 Renton to Bellevue - Moment Slab Barrier - Sign Bridge (9.90L)

Job No.: 650512  
Calc. By: WM

Section: Moment Slab  
Longitudinal Moment

Checked:

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## DESIGN SECTION

### ELEMENTS

Name	Library	Material Property
SE1: Moment Slab	User Library	MP1: C4 Es 4.3E3

#### Element coordinates for SE1: Moment Slab

Node	X-Y coords IN	Node	X-Y coords IN	Node	X-Y coords IN
1	128.0	15.0	6	1.5	53.0
2	52.319	23.671	7	-2.E-10	40.0
3	43.875	68.0	8	1.5008	38.999
4	1.875	68.0	9	0.0	25.999
5	-2.E-10	54.0	10	0.0	0.0
				11	5.875
				12	5.875
				13	15.375
				14	15.375
				15	128.0



Job: I-405 Renton to Bellevue - Moment Slab Barrier - Sign Bridge (9.90L)

Job No.: 650512  
Calc. By: WMSection: Moment Slab  
Longitudinal Moment

Checked:

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**REINFORCEMENT**

Ref	X-Y coords IN	Diam IN	Initial Strain	Material Property
1	4.12465	9.86564	0.5	0.0 MP2: Grade 60 E 2.9E4
2	3.625	3.25	0.5	0.0 MP2: Grade 60 E 2.9E4
3	18.5625	4.1875	0.875	0.0 MP2: Grade 60 E 2.9E4
4	124.812	4.1875	0.875	0.0 MP2: Grade 60 E 2.9E4
5	30.3681	4.1875	0.875	0.0 MP2: Grade 60 E 2.9E4
6	42.1736	4.1875	0.875	0.0 MP2: Grade 60 E 2.9E4
7	53.9792	4.1875	0.875	0.0 MP2: Grade 60 E 2.9E4
8	65.7847	4.1875	0.875	0.0 MP2: Grade 60 E 2.9E4
9	77.5903	4.1875	0.875	0.0 MP2: Grade 60 E 2.9E4
10	89.3958	4.1875	0.875	0.0 MP2: Grade 60 E 2.9E4
11	101.201	4.1875	0.875	0.0 MP2: Grade 60 E 2.9E4
12	113.007	4.1875	0.875	0.0 MP2: Grade 60 E 2.9E4
13	16.9375	10.0625	0.875	0.0 MP2: Grade 60 E 2.9E4

**LOAD DATA****SL1: Mlong  
Total Applied Forces**

Nominal	Service	Strength
X Moment = 0.0	X Moment = 0.0	X Moment = 1339.23 KIP.FT
Y Moment = 0.0	Y Moment = 0.0	Y Moment = 0.0 KIP.FT
Axial = 0.0	Axial = 0.0	Axial = 0.0 KIP
Y Shear = 0.0	Y Shear = 0.0	Y Shear = 149.12 KIP
X Shear = 0.0	X Shear = 0.0	X Shear = 0.0 KIP

FIND limiting capacity for: \* Not used \*

Job: I-405 Renton to Bellevue - Moment Slab Barrier - Sign Bridge (9.90L)

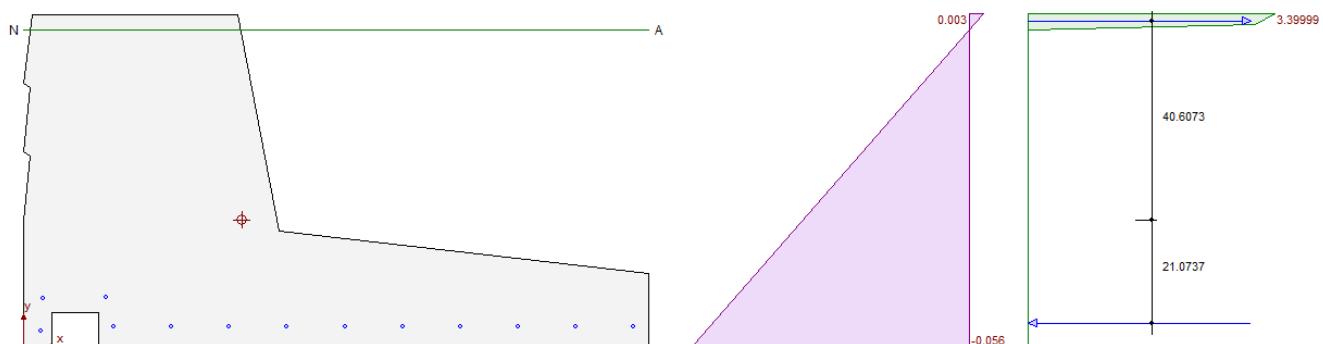
Job No.: 650512  
Calc. By: WM

Section: Moment Slab  
Longitudinal Moment

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#### ANALYSIS TO AASHTO LRFD ARTICLE 5.7.2.1 FOR STRENGTH LIMIT STATE DESIGN



Neutral axis fixed horizontal  
Neutral axis angle (applied loads) =  $0.0^\circ$  (clockwise from xx)  
depth in compression,  $c = 0.2868983$  FT

Resistance Factor used,  $\phi = 0.9$

#### Resistance Factor ( $\phi$ ) from Article 5.5.4.2.1

Type of force effect is:  
Tension-controlled reinforced concrete:  
 $\phi = 0.90$

#### MAXIMUM Strains:

Material Property	Strain	Stress KSI	location	
			x	y
C4 Es 4.3E3	0.003	3.3999932	43.875	68.0
Grade 60 E 2.9E4	-0.047486	-60.0	16.9375	10.0625

#### MINIMUM Strains:

Material Property	Strain	Stress KSI	location	
			x	y
C4 Es 4.3E3	-0.056254	0.0	0.0	0.0
Grade 60 E 2.9E4	-0.053422	-60.0	3.625	3.25



Job: I-405 Renton to Bellevue - Moment Slab Barrier - Sign Bridge (9.90L)

Job No.: 650512  
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Longitudinal Moment

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### LOADING relative to centroidal axes:

	Applied Loads	Required Strength	Spare Resistance	(Equilibrium Forces)
Mx KIP.FT	1339.23	1488.033	672.92119	2160.9545
My KIP.FT	0.0	0.0	n/a	-1410.9016
Ax KIP	0.0	0.0	n/a	-0.0314671
Spare resistance for X Moment - Positive = 672.92119 KIP.FT				
therefore limiting additional load = 672.921 * 0.9 = 605.629 KIP.FT				

### Check Minimum Reinforcement

The section is noncompression-controlled, therefore Article 5.7.3.3.2 applies for minimum reinforcement.

$$M_{cr} = \gamma_3 \cdot (\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe}) \cdot S_c \quad (5.7.3.3.2-1)$$

For Monolithic sections with no prestress,

$$M_{cr} = \gamma_3 \cdot \gamma_1 \cdot f_r \cdot S_c \quad (5.7.3.3.2-1)$$

$$\gamma_1 = 1.6$$

$$\gamma_3 = 0.67 \text{ for A615, Grade 60 reinforcement}$$

$$0.75 \text{ for A706, Grade 60 reinforcement}$$

$$f_r = 0.24/f'_c$$

$$= 0.48 \text{ KSI}$$

$$S_c = 70720.345 \text{ IN}^3$$

hence,

$$M_{cr} = 1.072 * 0.48 * 70720.3 = 3032.49 \text{ KIP.FT for A615}$$

$$M_{cr} = 1.2 * 0.48 * 70720.3 = 3394.58 \text{ KIP.FT for A706}$$

$M_r$  must be not less than the lesser of  $M_{cr}$  and  $1.33M_u$

### Summary of Internal Forces

	Force	Acting Height	
	KIP	IN	
compression force, $F_{cd}$			
- concrete	420.40199	66.5054	
- reinforcement (no compression reinforcement)			
- combined	420.40199	66.5054	
tension force, $F_{td}$			
- reinforcement	-420.4335	4.8245	
inner lever arm, $z = 66.5054 - 4.8245 = 61.680943 \text{ IN}$			

Note: Moments arising from eccentricity of axial loads are relative to the centroidal axis of the section.

The transformed section centroid is used.

Job: I-405 Renton to Bellevue - Moment Slab Barrier - Sign Bridge (9.90L)

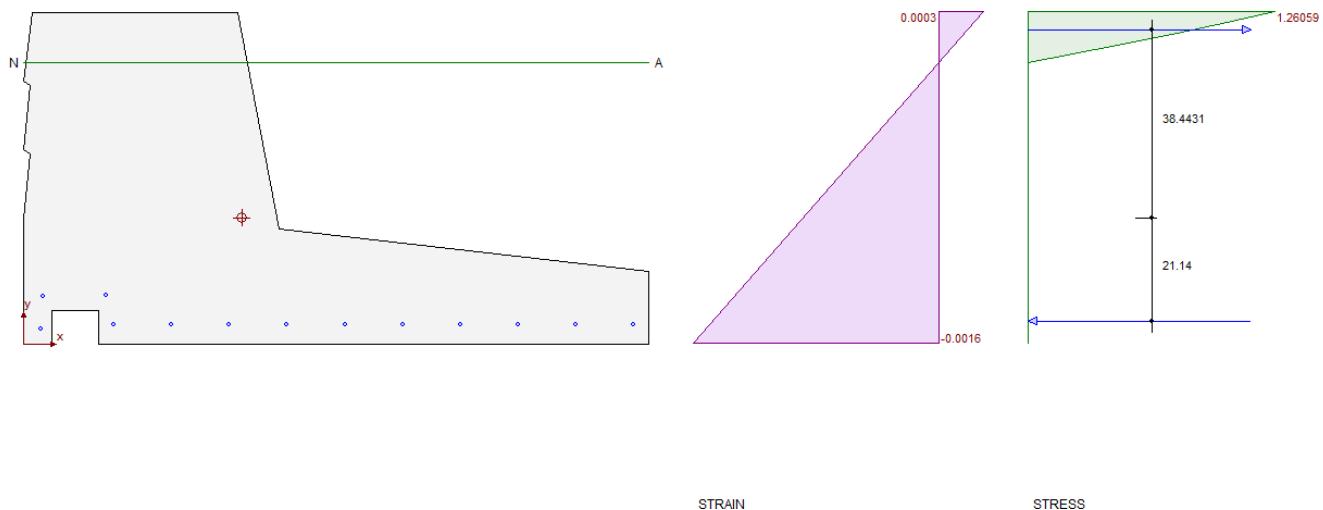
Job No.: 650512  
Calc. By: WM

Section: Moment Slab  
Longitudinal Moment

Checked:

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#### ANALYSIS TO AASHTO LRFD ARTICLE 5.7.2.1 FOR STRENGTH LIMIT STATE DESIGN



**WARNING** - This analysis assumes that all tension steel (Ast) and prestressing steel (Aps) are fully anchored in accordance with article 5.8.3.4.2.

All reinforcement is assumed to be parallel, and to be perpendicular to the cross section.

#### Analysis:

Shear Forces and associated Bending Moments  
Limit State - Strength I

#### Design Code:

AASHTO LRFD Bridge Design Specifications  
Seventh Edition with 2016 Interim Revisions

#### SUMMARY OF APPLIED LOAD EFFECTS

Moment $M_x$	1339.23 KIP.FT	Shear $V_y$	149.12 KIP
Moment $M_y$	0.0 KIP.FT	Shear $V_x$	0.0 KIP
Neutral Axis angle	0.0°		
Resolved Moment	1339.23 KIP.FT		
Resolved Shear	149.12 KIP		
Axial Load	0.0 KIP		



Job: I-405 Renton to Bellevue - Moment Slab Barrier - Sign Bridge (9.90L)

Job No.: 650512  
Calc. By: WM

Section: Moment Slab  
Longitudinal Moment

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## Article 5.8.3

### Transverse Shear Reinforcement

factored shear force  $V_u = 149.12$  KIP  
factored moment  $M_u = 1339.23$  KIP.FT

The factored moment at this section is positive, and so the following calculations relate to the section as defined

#### **Calculate effective shear depth, $d_v$ - Article 5.8.2.9**

overall depth of section  $h = 68.0$  IN  
compression fiber to tension force centroid  $d_e = 63.1755$  IN  
resistance moment of section (no axial load)  $M_r = 2160.89$  KIP.FT  
total tension force  $T = 420.433$  KIP  
distance from resultant tension to compression  $= M_r/T$   
 $= 5.13966$  FT  
 $0.9*d_e = 0.9*63.1755 = 56.858$  IN  
 $0.72*h = 0.72*68.0 = 48.96$  IN  
hence, effective shear depth,  $d_v = 61.676$  IN

#### **Critical section for shear - Article 5.8.3.2**

angle of compressive stress (see below)  $\theta = 36.0563^\circ$

$$0.5*d_v.\cot(\theta) = 0.5*61.676*1.37354  
= 42.3573$$
 IN

critical section for shear is larger of  $d_v$  and 42.3573 IN  
hence, critical section for shear:  
from face of support = 5.13966 FT



Job: I-405 Renton to Bellevue - Moment Slab Barrier - Sign Bridge (9.90L)

Job No.: 650512  
Calc. By: WM

Section: Moment Slab  
Longitudinal Moment

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### Determine $\beta$ and $\theta$ - Article 5.8.3.4

#### Shear stress on the concrete - 5.8.2.9-1

Resistance factor for shear [A5.5.4.2]  $\phi = 0.9$   
Vertical component of prestress at section  $V_p = 0.0$  KIP

$$v = \frac{V_u - \phi \cdot V_p}{\phi \cdot b_v \cdot d_v} = \frac{149.12 - 0.9 \cdot 0.0}{0.9 \cdot 4.32453 \cdot 5.13966} = 0.05177 \text{ KSI}$$

Factored axial force  $N_u = 0.0$  KIP  
Area of prestress steel on tension side  $A_{ps} = 0.0$  IN<sup>2</sup>  
Area of concrete on flexural tension side  $A_c = 3052.76$  IN<sup>2</sup>  
Area of non-prestress steel on tension side  $A_s = 7.00722$  IN<sup>2</sup>  
Modulus of elasticity of tendons  $E_p = 0.0$  KSI  
Modulus of elasticity of non-prestress steel  $E_s = 29000.0$  KSI  
Modulus of elasticity of concrete  $E_c = 4266.2$  KSI  
 $f_{po} = 0.7 \cdot f_{pu}$   
 $= 0.7 \cdot 0.0$   
 $= 0.0$  KSI  
 $|V_u - V_p| \cdot d_v = 766.427$  KIP.FT  
 Factored moment (not less than  $|V_u - V_p| \cdot d_v$ )  $|M_u| = 1339.23$  KIP.FT

#### General procedure: (Article 5.8.3.4.2)

Strain in reinforcement:

$$\varepsilon_s = \frac{\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}}{E_s A_s + E_p A_{ps}} \quad (\text{Eq. 5.8.3.4.2-4})$$

$$\varepsilon_s = \frac{\left[ \frac{1339.2}{5.1397} + 0.5 \cdot 0.0 + |149.12 - 0.0| - 0.0 \cdot 0.0 \right]}{2.9E4 \cdot 7.0072 + 0.0 \cdot 0.0}$$

$$= 0.002$$

with at least the minimum amount of shear reinforcement:

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)} = \frac{4.8}{(1 + 750 \cdot 0.002)} \quad (\text{Eq. 5.8.3.4.2-1})$$

$$= 1.91078$$



Job: I-405 Renton to Bellevue - Moment Slab Barrier - Sign Bridge (9.90L)

Job No.: 650512  
Calc. By: WM

Section: Moment Slab  
Longitudinal Moment

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with no shear reinforcement:

$$s_x = 0.0 \text{ FT}$$

maximum aggregate size:  $a_g = 1.0 \text{ IN}$

$$s_{xe} = s_x * \frac{1.38}{a_g + 0.63} = 0.0 * \frac{1.38}{1.0 + 0.63} = 0.0 \text{ FT}$$

12.0IN  $\leq s_{xe} \leq 80.0\text{IN}$

$$s_{xe} = 12.0 \text{ IN}$$

$$\begin{aligned} \beta &= \frac{4.8}{(1 + 750\varepsilon_s)} * \frac{51}{(39 + s_{xe})} && (\text{Eq. 5.8.3.4.2-2}) \\ &= \frac{4.8}{(1 + 750*0.002)} * \frac{51}{(39 + 12.0)} \\ &= 1.91078 \end{aligned}$$

$$\begin{aligned} \theta &= 29 + 3500\varepsilon_s && (\text{Eq. 5.8.3.4.2-3}) \\ &= 29 + 3500* 0.002 \\ &= 36.0563^\circ \end{aligned}$$

## Nominal Shear Resistance

### Article 5.8.3.3

inclination of the transverse reinforcement,  $\alpha = 90.0^\circ$

$$\text{nominal shear resistance } V_n = V_c + V_s + V_p \quad (5.8.3.3-1)$$

with at least the minimum amount of shear reinforcement:

$$\begin{aligned} V_c &= 0.0316 * \beta \lambda (\sqrt{f'_c}) * b_v * d_v && (5.8.3.3-3) \\ &= 0.0316 * 1.9108 * 1.0 * (\sqrt{4.0}) * 51.894 * 61.676 \\ &= 386.5 \text{ KIP} \end{aligned}$$

with no shear reinforcement:

$$\begin{aligned} V_c &= 0.0316 * \beta \lambda (\sqrt{f'_c}) * b_v * d_v && (5.8.3.3-3) \\ &= 0.0316 * 1.9108 * 1.0 * (\sqrt{4.0}) * 51.894 * 61.676 \\ &= 386.5 \text{ KIP} \end{aligned}$$

$$\begin{aligned} V_p &= 0.0 \text{ KIP} \\ V_s &\text{ depends upon the reinforcement present.} \end{aligned}$$



Job: I-405 Renton to Bellevue - Moment Slab Barrier - Sign Bridge (9.90L)

Job No.: 650512  
Calc. By: WM

Section: Moment Slab  
Longitudinal Moment

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## Maximum Shear Force

### Articles 5.8.2.4 and 5.8.3.3

Maximum value of shear resistance with no transverse reinforcement

$$\begin{aligned} V_{r,no \ links} &= 0.5*\phi.(V_c + V_p) & (5.8.2.4-1) \\ &= 0.5*0.9*(386.503+0.0) \\ &= 173.927 \text{ KIP} \end{aligned}$$

Maximum value of shear resistance with transverse reinforcement

$$\begin{aligned} V_{n,max} &= 0.25 * f'_c.b_v.d_v + V_p & (5.8.3.3-2) \\ &= 0.25 * 4.0*51.8944*61.676 + 0.0 \\ &= 3200.64 \text{ KIP} \end{aligned}$$

$$\begin{aligned} V_{r,max} &= \phi.V_{n,max} \\ &= 0.9*3200.64 \\ &= 2880.57 \text{ KIP} \end{aligned}$$

## Transverse Reinforcement Requirements

inclination of the transverse reinforcement,  $\alpha = 90.0^\circ$

minimum yield strength of transverse rft  $f_y = 60.0 \text{ KSI}$

### Minimum Transverse Reinforcement (article 5.8.2.5):

$$\frac{A_v}{s} \geq \frac{0.0316*\lambda*\sqrt{f'_c.b_v}}{f_y}$$
$$\begin{aligned} &= 0.0316*1.0*\sqrt{4.0*51.8944}/60.0 \\ &= 0.0547 \text{ IN}^2/\text{IN} \end{aligned}$$



Job: I-405 Renton to Bellevue - Moment Slab Barrier - Sign Bridge (9.90L)

Job No.: 650512  
Calc. By: WM

Section: Moment Slab  
Longitudinal Moment

Checked:

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### Shear resistance (article 5.8.3.3) with minimum reinforcement (article 5.8.3.3):

$$V_n = V_c + V_s + V_p$$

$$V_s = A_v/s \cdot f_y \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha) \quad (5.8.3.3-4)$$

$$\begin{aligned} V_{s,min} &= 0.0547 * 60.0 * 61.676 * 1.37354 \\ &= 278.034 \text{ KIP} \end{aligned}$$

therefore,

$$\begin{aligned} V_{n,min} &= 386.513 + 278.034 + 0.0 \\ &= 664.547 \text{ KIP} \\ \phi \cdot V_{n,min} &= 0.9 * 664.547 \\ &= 598.092 \text{ KIP} \end{aligned}$$

### Additional transverse reinforcement for shear (article 5.8.3.3):

$V_u$  is less than  $\phi V_{n,zero}$ , therefore no reinforcement is required.

### Longitudinal Reinforcement - Article 5.8.3.5

Non-prestressed reinforcing steel:

$$\begin{aligned} \text{Area on flexural tensile side } A_s &= 7.00722 \text{ IN}^2 \\ \text{yield stress } f_y &= 60.0 \text{ KSI} \\ \text{tensile capacity} &= 7.00722 * 60.0 \\ &= 420.433 \text{ KIP} \end{aligned}$$

Prestressing steel:

$$\text{Area on flexural tensile side } A_{ps} = 0.0 \text{ IN}^2$$

$$\begin{aligned} \text{Total tensile capacity} &= 420.433 + 0.0 \\ &= 420.433 \text{ KIP} \end{aligned}$$

Equation 5.8.3.5-1 right hand side:

$$\begin{aligned} M_u / (d_v \cdot \phi) &= 1339.23 / (5.13966 * 0.9) \\ &= 289.52 \text{ KIP} \\ 0.5 * N_u / \phi &= 0.5 * 0.0 / 0.75 \\ &= 0.0 \text{ KIP} \\ V_u / \phi - 0.5 * V_s - V_p &= 149.12 / 0.9 - 0.5 * 0.0 - 0.0 \\ &= 165.689 \\ * \cot(\theta) &= 165.689 * 1.37354 \\ &= 227.581 \text{ KIP} \end{aligned}$$

where  $V_s$  is taken from equation 5.8.3.3-4 but not greater than  $V_u / \phi$

$$\begin{aligned} V_u / \phi &= 149.12 / 0.9 \\ &= 165.689 \text{ KIP} \\ A_v/s \cdot f_y \cdot d_v \cdot \cot(\theta) &= 0.0 * 60.0 * 5.13966 * 1.37354 \\ &= 0.0 \text{ KIP} \end{aligned}$$



Job: I-405 Renton to Bellevue - Moment Slab Barrier - Sign Bridge (9.90L)

Job No.: 650512  
Calc. By: WM

Section: Moment Slab  
Longitudinal Moment

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$$289.52 + 0.0 + 227.581 = 517.1 \text{ KIP}$$

which is greater than 420.433, so Equation 1 is not satisfied, therefore additional longitudinal reinforcement is required.

Area of fully developed non-prestressed reinforcing steel required on flexural tension side of member is given by:

$$A_s \cdot f_y = 517.1 - 420.433 = 96.6667 \text{ KIP}$$

hence additional area required,

$$\begin{aligned} &= 96.6667 / 60.0 \\ &= 1.61111 \text{ IN}^2 \end{aligned}$$



## **RETAINING WALL 09.90L - A & B**

*3.0 —Barrier Support Light Pole Design*

<b>PARSONS</b>		MADE BY		DATE	CHK BY	DATE
		B. Christophersen		11/9/2021	W. Metwally	11/19/2021
Job Number 649974	WBS Number 00531	TITLE		I-405 Renton to Bellevue		
				DESIGN OF BARRIER REINFORCEMENT FOR LIGHT POLE SUPPORT		

**[A] PURPOSE**

- Design barrier that supports the lightpole.
- The load demands provided in this calculation will NOT be used for moment slab design. Vehicle impact will govern the barrier/slab design.

**[B] REFERENCES:**

Acronym	Source
AASHTO	AASHTO LRFD Bridge Design Specifications (8th Edition)
BDM	WSDOT Bridge Design Manual M23-50.18 (2018 Edition)
ACI 318-14	Building Code Requirement for Structural Concrete, 2014

**[C] SUMMARY AND CONCLUSION**

Pole

OD at Bott.	OD at Top	t	Height
in	in	in	ft
13.5	10.5	0.3125	40.00

**[D] MATERIALS:**

$$\gamma_{c,E} = \begin{cases} 150 \\ 145 \end{cases} \text{ lb/ft}^3 \quad \text{(reinforced concrete unit weight for capacity analysis)}$$

[BDM Table 3.8-1]  
[AASHTO-Table 3.5.1-1]

Unconfined Concrete

f <sub>c</sub>	K <sub>1</sub>	E <sub>c</sub>	f <sub>r</sub>	ε <sub>cu</sub>	v <sub>c</sub>	G <sub>c</sub>
ksi	#	ksi	ksi	#	#	ksi
4.0	1.00	4,266	0.48	0.003	0.2	1,778

f<sub>c</sub> = {concrete compressive strength }

$$E_c = 120,000 K_1 \gamma_c^2 f_c^{0.33}$$

f<sub>r</sub> = 0.24√(f<sub>c</sub>) {concrete tension capacity for this analysis}

ε<sub>cu</sub> = {maximum unconfined concrete strain}

v<sub>c</sub> = {poisson's ratio for concrete}

$$G_c = E_c / (2*(1+v_c)) \quad \text{(concrete shear modulus)}$$

[AASHTO-5.4.2.4-1]

[AASHTO-5.4.2.6]

[AASHTO-5.6.2.1]

[AASHTO 5.4.2.5]

ASTM A706 Grade 60 Reinforcing Steel

f <sub>y</sub>	f <sub>u</sub>	E <sub>s</sub>	ε <sub>y</sub>
ksi	ksi	ksi	#
60	80	29,000	0.0021

f<sub>y</sub> = {minimum yield strength}

f<sub>u</sub> = {minimum tensile strength}

E<sub>s</sub> = {steel modulus of elasticity}

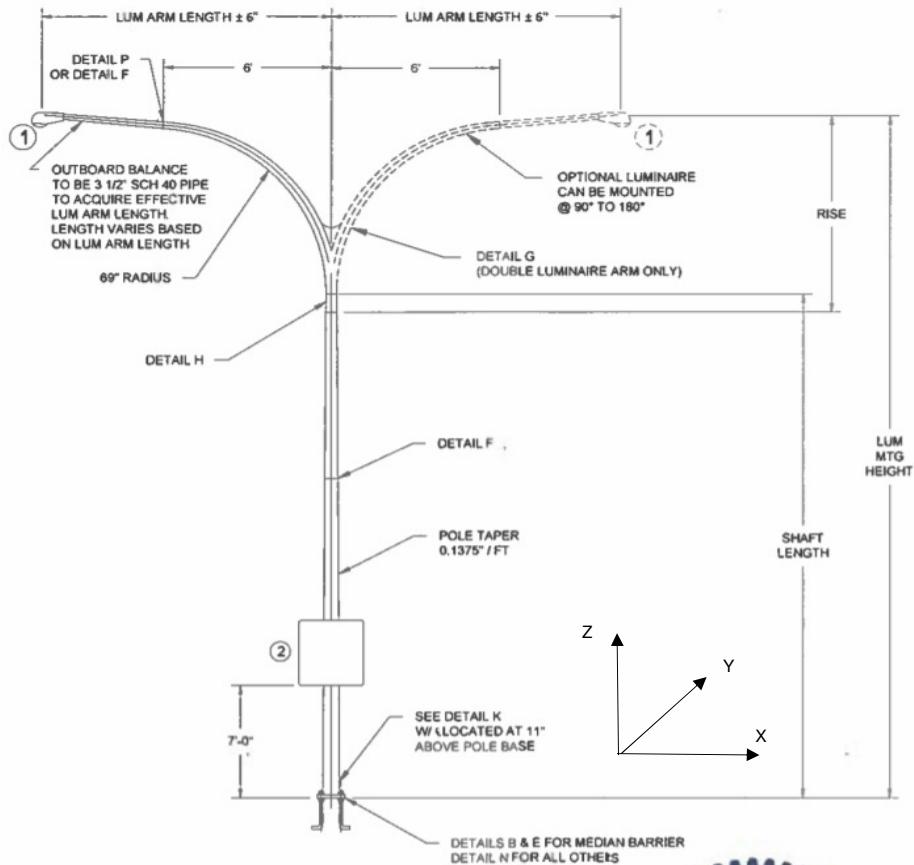
ε<sub>y</sub> = {nominal yield strain}

[ASTM A706-16 Table A1.2]

[ASTM A706-16 Table A1.2]

[AASHTO 5.4.3.2]

<b>PARSONS</b>		MADE BY		DATE	CHK BY	DATE
		B. Christophersen		11/9/2021	W. Metwally	11/19/2021
Job Number 649974	WBS Number 00531	TITLE		I-405 Renton to Bellevue		
				DESIGN OF BARRIER REINFORCEMENT FOR LIGHT POLE SUPPORT		



## [E] GEOMETRY

Base of Pole to Top of Foundation = **44** in  
 Pole Shape: **Round**



## [F] LOADS

- Loads at the top of the barrier is taken from drawing ILD-408

### NOTES:

1. THE FOUNDATION HAS BEEN DESIGNED ACCORDING TO THE AASHTO LRFD SPECIFICATIONS FOR STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS, LUMINAIRES AND TRAFFIC SIGNALS, FIRST EDITION, 2015. ULTIMATE WIND SPEED IS 120 MPH.

2. THE FOUNDATION HAS BEEN DESIGNED FOR THE FOLLOWING POLE BASE FORCES:

BENDING MOMENT: 38,023 FT-LB

TORSION: 1,389 FT-LB

SHEAR FORCE: 1,700 LB

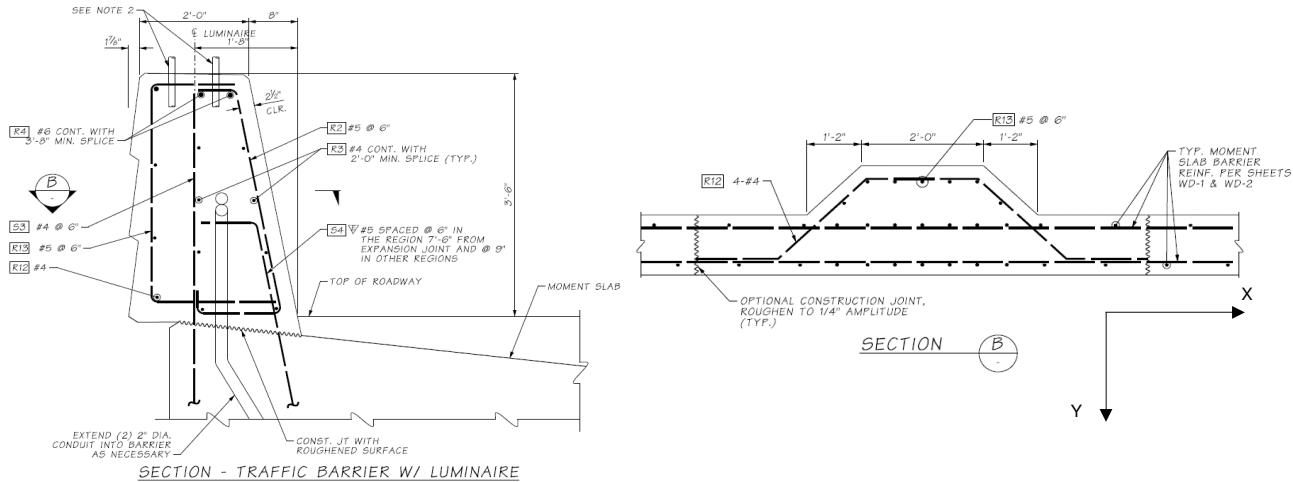
AXIAL FORCE: 1,715 LB

3. FOUNDATIONS FOR POLES WITH FORCES GREATER THAN ONE OR MORE OF THE FORCES LISTED ABOVE SHALL REQUIRE SPECIAL DESIGN.

Top of Barrier				Top of Moment Slab			
Axial $P_u$	Shear $V_u$	Moment $M_u$	Torsion $T_u$	Axial $P_u$	Shear $V_u$	Moment $M_u$	Torsion $T_u$
k	k	k-in	k-in	k	k	k-in	k-in
1.72	1.7	456	16.67	1.72	1.7	532	16.67

<b>PARSONS</b>		MADE BY		DATE	CHK BY	DATE
		B. Christophersen		11/9/2021	W. Metwally	11/19/2021
Job Number 649974	WBS Number 00531	TITLE		I-405 Renton to Bellevue		
				DESIGN OF BARRIER REINFORCEMENT FOR LIGHT POLE SUPPORT		

### [G] BARRIER REINFORCEMENT DESIGN



Location	Loading	Bar Size	Bar Diameter, d	Bar Area, $A_s$	Number of Bars	$A_{s,tot}$	Bar Spacing
Text	Text	#	in	in <sup>2</sup>	#	in <sup>2</sup>	in
Barrier	+M <sub>x</sub>	# 4	0.50	0.20	3	0.6	6
Barrier	-M <sub>x</sub>	# 5	0.63	0.31	5	1.55	6
Slab	+M <sub>x</sub>	# 4	0.50	0.20	5	1	6
Slab	-M <sub>x</sub>	# 5	0.63	0.31	5	1.55	6

### MATERIALS

f <sub>y</sub>	f <sub>c</sub>	$\beta_1$	$\varepsilon_{cl}$	$\varepsilon_{dl}$	$\varepsilon_{cu}$
ksi	ksi	#	#	#	#
60	4	0.85	0.002	0.005	0.003

### GEOMETRY

Depth of Barrier at Top, h <sub>top</sub>	Depth of Barrier at Bottom, h <sub>bot</sub>	Depth of Barrier at Slab, h <sub>slab</sub>	Width of Square Barrier Extension, b	Barrier Clear Cover, C
in	in	in	in	in
24	32	18	24	2.5

### FLEXURE DESIGN

Location	Loading	M <sub>u</sub>
Text	Text	k-in
Barrier	+M <sub>x</sub>	456
Barrier	-M <sub>x</sub>	456
Slab	+M <sub>x</sub>	532
Slab	-M <sub>x</sub>	532

<b>PARSONS</b>		MADE BY			DATE		CHK BY		DATE				
		B. Christophersen			11/9/2021		W. Metwally		11/19/2021				
Job Number	WBS Number	TITLE		I-405 Renton to Bellevue									
649974	00531			DESIGN OF BARRIER REINFORCEMENT FOR LIGHT POLE SUPPORT									

Location	Loading	d <sub>s</sub>	c	a	c/d <sub>s</sub>	Check c/d <sub>s</sub> Limit	M <sub>n</sub>	ε <sub>t</sub>	ϕ	ϕM <sub>n</sub>	D/C
Text	Text	in	in	in	#	#	k-in	#	#	k-in	#
Barrier	+M <sub>x</sub>	21.25	0.52	0.44	0.024	OK	757	0.120	0.9	681	0.67
Barrier	-M <sub>x</sub>	21.19	1.34	1.14	0.063	OK	1,917	0.044	0.9	1726	0.26
Slab	+M <sub>x</sub>	15.25	0.87	0.74	0.057	OK	893	0.050	0.9	804	0.66
Slab	-M <sub>x</sub>	15.19	1.34	1.14	0.088	OK	1,359	0.031	0.9	1223	0.43

$$c = A_{s,tot} * f_y / (0.85 * \beta_1 * f'_c * b) \quad [\text{AASHTO 5.6.3.1.1}]$$

$$a = c * \beta_1 \quad [\text{AASHTO 5.6.2.2}]$$

$$M_n = A_s f_y * (d_{s,max} - a / 2) \quad [\text{AASHTO 5.6.3.2.2}]$$

$$\varepsilon_t = \varepsilon_{cu} * (d_t - c) / c \quad \{\text{net tensile strength in extreme tension steel at nominal resistance}\}$$

$$\phi = \max (0.75, \min (0.90, 0.75 + 0.15 * (\varepsilon_t - \varepsilon_{cl}) / (\varepsilon_{tl} - \varepsilon_{cl}))) \quad [\text{strain compatibility}]$$

#### Check minimum wall flexural reinforcement [AASHTO 5.6.3.3].

$$\lambda = \boxed{1.00} \quad \{\text{concrete density modification factor}\} \quad [\text{AASHTO 5.4.2.8}]$$

Location	Loading	f <sub>r</sub>	γ <sub>1</sub>	γ <sub>2</sub>	γ <sub>3</sub>	f <sub>cpe</sub>	S <sub>c</sub>	M <sub>cr</sub>	M <sub>min</sub>	ϕM <sub>n</sub>	D/C
Text	#	ksi	#	#	#	ksi	in <sup>3</sup>	k-in	k-in	k-in	#
Barrier	+M <sub>x</sub>	0.480	1.60	1.00	0.75	0.00	2304	111	110.6	681.4	0.16
Barrier	-M <sub>x</sub>	0.480	1.60	1.00	0.75	0.00	2304	111	110.6	1725.7	0.06
Slab	+M <sub>x</sub>	0.480	1.60	1.00	0.75	0.00	1296	62	62.2	803.6	0.08
Slab	-M <sub>x</sub>	0.480	1.60	1.00	0.75	0.00	1296	62	62.2	1223.5	0.05

$$f_r = 0.24 \lambda \sqrt{f_c} \quad \{\text{concrete modulus of rupture}\} \quad [\text{AASHTO 5.4.2.6}]$$

$$\gamma_1 = \{\text{flexural cracking variability factor}\} \quad [\text{AASHTO 5.6.3.3}]$$

$$\gamma_2 = \{\text{prestress variability factor}\} \quad [\text{AASHTO 5.6.3.3}]$$

$$\gamma_3 = \{\text{ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement}\} \quad [\text{AASHTO 5.6.3.3}]$$

$$f_{cpe} = \{\text{compressive stress in concrete due to effective prestress force only}\} \quad [\text{AASHTO 5.6.3.3}]$$

$$S_c = b t_{wall}^2 / 6 \quad \{\text{section modulus per unit length of wall}\}$$

#### 5.6.3.3—Minimum Reinforcement

Unless otherwise specified, at any section of a noncompression-controlled flexural component, the amount of prestressed and nonprestressed tensile reinforcement shall be adequate to develop a factored flexural resistance,  $M_r$ , greater than or equal to the lesser of the following:

- 1.33 times the factored moment required by the applicable strength load combination specified in Table 3.4.1-1;

$$M_{cr} = \gamma_3 \left[ \left( \gamma_1 f_r + \gamma_2 f_{cpe} \right) S_c - M_{dn} \left( \frac{S_c}{S_{nc}} - 1 \right) \right]$$

<b>PARSONS</b>		MADE BY		DATE	CHK BY	DATE
		B. Christophersen		11/9/2021	W. Metwally	11/19/2021
Job Number	WBS Number	TITLE	I-405 Renton to Bellevue			
649974	00531		DESIGN OF BARRIER REINFORCEMENT FOR LIGHT POLE SUPPORT			

### SHEAR DESIGN

V <sub>u,Barrier</sub>	V <sub>u,Slab</sub>
k/ft	k/ft
1.70	1.70

Location	Loading	b <sub>v</sub>	h	d <sub>s</sub>	d <sub>v</sub>
Text	#	in	in	in	in
Barrier	+M <sub>x</sub>	24.00	24	21.25	19.13
Barrier	-M <sub>x</sub>	24.00	24	21.19	19.07
Slab	+M <sub>x</sub>	24.00	18	15.25	13.73
Slab	-M <sub>x</sub>	24.00	18	15.19	13.67

b<sub>v</sub> = {effective width of section measured parallel to neutral axis}

h = {thickness of the section in the direction of loading}

d<sub>s</sub> = {depth to the center of flexural reinforcement in the direction of loading}

d<sub>v</sub> = MAX( 0.72\*h, 0.9\*d<sub>s</sub>) [AASHTO 5.7.2.8]

ϕ<sub>s</sub> =

0.9

[AASHTO 5.5.4.2]

Location	Loading	ε	β	θ	V <sub>c</sub>	Is Transverse Reinforcement Required?
Text	#	#	#	Degrees	k/ft	
Barrier	+M <sub>x</sub>	-	2	45	58.02	Transverse Reinforcement Not Required
Barrier	-M <sub>x</sub>	-	2	45	57.85	Transverse Reinforcement Not Required
Slab	+M <sub>x</sub>	-	2	45	41.64	Transverse Reinforcement Not Required
Slab	-M <sub>x</sub>	-	2	45	41.47	Transverse Reinforcement Not Required

ε<sub>s</sub> = {net longitudinal tensile strain at the centroid of tension reinforcement}

[AASHTO 5.7.3.4.2-4]

β = {factor indicating ability of diagonally cracked concrete to transmit tension and shear}

[AASHTO 5.7.3.4.1]

θ = {angle of inclination of diagonal compressive stresses}

[AASHTO 5.7.3.4.1]

V<sub>c</sub> = 0.0316 \* β \* √f<sub>c</sub> \* b<sub>v</sub> \* d<sub>v</sub> {shear resistance of concrete}

[AAHSTO 5.7.3.3-3]



## RETAINING WALL 09.90L - A & B

4.0 — *Retaining Barrier Design (Wall 9.85L)*

 PARSONS		MADE BY	DATE	CHK BY	DATE
		B. Christophersen	11/23/2021	N. Ala	11/24/2021
Job Number	WBS Number	TITLE	WSDOT I-405 - Renton to Bellevue		
650512	00515		Soil Retaining Barrier Design		

#### [A] BASIS

- To evaluate the load demands on the retaining barrier and design adequate barrier reinforcement.

#### [B] REFERENCES

Acronym	Source
AASHTO	AASHTO LRFD Bridge Design Specifications 8th Edition
BDM	WSDOT Bridge Design Manual (M23-50.19) - July 2019
GEM	Wall Package - Retaining Wall 9.85L Geotechnical Design Memo

#### [C] DESIGN NOTES AND PARAMETERS

1. The barrier will be evaluated during Strength I, Service I, Extreme I, and Extreme II load combinations.
2. The barrier must be adequately stable during the Strength I, Service I, and Extreme I load combinations. Extreme II will be ignored for stability calculations because a vehicle impact will be supported by the barrier in either direction of the impact and by engineering judgement, the barrier will not overturn or slide into the adjacent hill.

#### [D] MATERIAL PROPERTIES

$\gamma_c =$	145.0 pcf	{plain concrete unit weight for loads and models}	[BDM Table 3.8.1]
$\gamma_c =$	150.0 pcf	{reinforced concrete unit weight for modulus of elasticity}	[BDM 5.1.1D]
$\gamma_{rc} =$	155.0 pcf	{reinforced concrete unit weight for loads and models}	[BDM Table 3.8.1]

Plain Concrete									
Elements	$f_c$	$K_1$	$E_c$	$f_{ce}$	$E_{ce}$	$\alpha_{TU}$	$v$	$G_c$	$G_{ce}$
Text	ksi	#	ksi	ksi	ksi	°F <sup>-1</sup>	#	ksi	ksi
Retaining Barrier	4.00	1.00	4266	5.20	4967	6.00E-06	0.20	1778	2070

$f_c$  = {concrete compressive strength }

$K_1$  = {correction factor for source of aggregate}

$E_{rc} = 120000 K_1 (\gamma_{rc})^2 (f_c)^{0.33}$  {concrete modulus of elasticity} [AASHTO 5.4.2.4]

$f_{ce,e} = 1.3 f_c$  {expected concrete compressive strength} [SGS 8.4.4-1]

$E_{rc,e} = 120000 K_1 (\gamma_{rc})^2 (f_{ce,e})^{0.33}$  {expected concrete modulus of elasticity} [AASHTO 5.4.2.4]

$\alpha_{TU} = \{\text{coefficient of thermal expansion}\}$  [DCM 8.4.2.I.3]

$v = \{\text{poisson's ratio}\}$  [AASHTO 5.4.2.5]

$G_c = E_{rc} / (2*(1+v))$  {concrete shear modulus}

$G_{ce} = E_{rc,e} / (2*(1+v))$  {concrete expected shear modulus}

 PARSONS		MADE BY	DATE	CHK BY	DATE
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650512	00515		Soil Retaining Barrier Design		

ASTM A706 Grade 60 Reinforcing Steel							
Bar Size	f <sub>y</sub>	f <sub>u</sub>	E <sub>s</sub>	f <sub>ye</sub>	f <sub>ue</sub>	ε <sub>y</sub>	ε <sub>ye</sub>
#	ksi	ksi	ksi	ksi	ksi	#	#
All	60	80	29000	68	95	0.0021	0.0023

f<sub>y</sub> = {minimum yield strength}

[ASTM A706-16 Table A1.2]

f<sub>u</sub> = {minimum tensile strength}

[ASTM A706-16 Table A1.2]

E<sub>s</sub> = {steel reinforcement modulus of elasticity}

[AASHTO 5.4.3.2]

f<sub>ye</sub> = {expected minimum yield strength}

[SGS Table 8.4.2-1]

f<sub>ue</sub> = {expected minimum tensile strength}

[SGS Table 8.4.2-1]

ε<sub>y</sub> = f<sub>y</sub> / E<sub>s</sub> {nominal yield strain}

[SGS Table 8.4.2-1]

ε<sub>ye</sub> = {expected yield strain}

ε <sub>tl</sub> =	0.005
ε <sub>cl</sub> =	0.002
ε <sub>c</sub> =	0.003

{tension-controlled reinf. steel strain limit}  
{compression-controlled reinf. steel strain limit}  
{maximum usable concrete compression strain}

[AASHTO 5.6.2.1]

[AASHTO 5.6.2.1]

[AASHTO 5.6.2.1]

Soil						
Soil Type	γ <sub>s</sub>	ϕ <sub>soil</sub>	tan δ = tan ϕ	K <sub>a</sub>	K <sub>AE</sub>	K <sub>p</sub>
#	pcf	deg	-	#	#	#
Fill	125.0	36	0.73	0.35	0.79	6.00

γ<sub>s</sub> = {soil unit weight}

ϕ<sub>soil</sub> = {internal friction angle of soil}

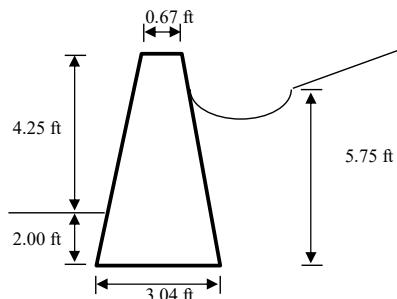
tan δ = {Coefficient of friction between soil and bottom of footing} = tan ϕ for cast in place concrete against soil

[BDM 7.7.4 C]

 PARSONS		MADE BY	DATE	CHK BY	DATE
		B. Christophersen	11/23/2021	N. Ala	11/24/2021
Job Number	WBS Number	TITLE	WSDOT I-405 - Renton to Bellevue		
650512	00515		Soil Retaining Barrier Design		

[E] GEOMETRY

Height above Roadway	Embedment Depth	Length	Top Thickness	Bottom Thickness	Soil Depth Behind Barrier	Wall Slope
ft	ft	ft	ft	ft	ft	Run/Rise
4.25	2.00	1.00	0.67	3.04	5.75	0.19



[F] GLOBAL STABILITY

D / C
CHECK BEARING STRESS: 0.29
CHECK SLIDING: 0.67
CHECK OVERTURNING: 0.98

 PARSONS		MADE BY	DATE	CHK BY	DATE	
		B. Christophersen	11/23/2021	N. Ala	11/24/2021	
Job Number	WBS Number	TITLE	WSDOT I-405 - Renton to Bellevue			
650512	00515		Soil Retaining Barrier Design			

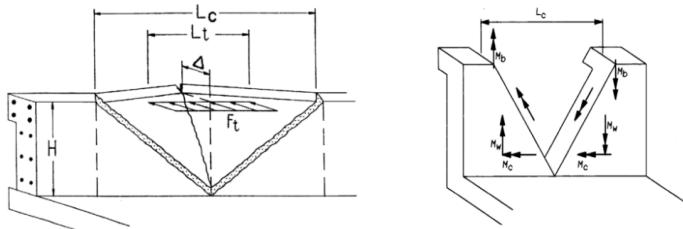
[G] LOADING

Barrier DC								
Element	Length	Height	Top Thickness	Bottom Thickness	Volume	Weight	Eccentricity CL Ftg	Moment CL Ftg
Text	ft	ft	ft	ft	ft <sup>3</sup> / ft	kip / ft	ft	k-ft / ft
Barrier	1.0	6.25	0.67	3.04	11.6	1.80	0.00	0.00

Lateral Soil Forces EH							
Load	L	H	P <sub>bot</sub>	P <sub>top</sub>	F <sub>long</sub>	y <sub>bot</sub>	M <sub>bot</sub>
Text	ft	ft	ksf	ksf	kip / ft	ft	k-ft / ft
EH <sub>a</sub>	1.00	5.8	0.25	0.00	0.72	1.92	1.39
EH <sub>p</sub>	1.00	2.0	1.50	0.00	-1.50	0.67	-1.00
EH <sub>p, CT, MID</sub>	1.00	3.8	2.81	0.00	5.27	1.25	6.59
EH <sub>p, CT, END</sub>	1.00	0.0	0.01	0.00	0.00	0.00	0.00
EH <sub>AE</sub>	1.00	5.8	0.57	0.00	1.63	1.92	3.13

- Ignore top 2ft of soil  
- Assume end of barrier has minimal soil behind wall

- Lateral vehicle impact loading is applied along length L<sub>t</sub> but the portion of barrier that contributes to resisting the load is the length L<sub>c</sub>. Length L<sub>c</sub> is determined per AASHTO A13.3.1 using the yield line failure pattern and the vertical/longitudinal moment capacities of the barrier.



Lateral Vehicle Impact Forces											
Load	Lateral Test Level	F <sub>t</sub>	L <sub>t</sub>	Location	Assumed L <sub>c</sub>	M <sub>w</sub>	M <sub>c</sub>	L <sub>c</sub>	H <sub>e</sub>	F <sub>long</sub>	M <sub>bot</sub>
Text	Text	k	ft	Text	ft	k-ft	k-ft	ft	in	k / ft	k-ft / ft
					Middle	9.97	287.31	222.74	9.97	32.00	-5.42
				Ends	5.07	287.31	225.88	5.07	-10.65	-14.44	

F<sub>t</sub> = {transverse vehicle impact load}

L<sub>t</sub> = {longitudinal length of distribution of F<sub>t</sub>}

M<sub>w</sub> = {moment capacity of the longitudinal reinforcement within the barrier}

M<sub>c</sub> = {moment capacity of the vertical reinforcement within the barrier accounting for the assumed L<sub>c</sub>}

L<sub>c</sub> = {critical length or yield line failure pattern}

H<sub>e</sub> = {height above roadway surface where the impact force is applied}

 PARSONS		MADE BY	DATE	CHK BY	DATE
		B. Christophersen	11/23/2021	N. Ala	11/24/2021
Job Number	WBS Number	TITLE	WSDOT I-405 - Renton to Bellevue		
650512	00515		Soil Retaining Barrier Design		

Lateral Inertial Forces						
Load	A <sub>s</sub>	k <sub>h</sub>	W	F <sub>long</sub>	y <sub>bot</sub>	M <sub>bot</sub>
Text	g	g	k / ft	k / ft	ft	k-ft / ft
IR <sub>DC</sub>	0.50	0.25	1.80	0.45	2.46	1.10

Vertical Soil Forces EV							
Load	L	B	H	V	P	x <sub>fg,cl</sub>	M <sub>fg,cl</sub>
Text	ft	ft	ft	ft <sup>3</sup> / ft	kip / ft	ft	k-ft / ft
EV	1.00	1.19	5.75	3.41	0.43	-1.13	-0.48

#### [H] LOAD FACTORS

Load Factors							
Limit State	DC max	DC min	EV min	EH max	EH min	EQ	CT
Strength I	1.25	0.90	1.00	1.50 Active	0.90 Active	0.00	0.00
				1.00 Passive	1.00 Passive		
Service I	1.00	-	1.00	1.00	-	0.00	0.00
Extreme Event I	1.00	-	1.00	1.00	-	1.00	0.00
Extreme Event II	1.00	-	1.00	1.00	-	-	1.00

#### [I] BARRIER EXTERNAL STABILITY

##### 1. Check footing eccentricity at service and strength limit states [AASHTO 10.6.3.4].

- Eccentricity limits are only applicable at the Strength Limit State per [AASHTO 11.6.2 & 11.6.3], however Service overturning moments and vertical force effects are still summarized below.

- Per [AASHTO 11.6.3.5], passive resistance shall be neglected in stability computations, unless the base of the wall extends below the depth of maximum scour, freeze-thaw, or other disturbances. Where passive resistance is utilized to ensure adequate wall stability, the calculated passive resistance of soil in front of conventional walls shall be sufficient to prevent unacceptable forward movement of the wall. Passive resistance will be considered for extreme events since the roadway will ensure that the barrier won't move on the roadway side and the soil will ensure the barrier won't move on the back face.

- Per [AASHTO 11.6.5.1], 50% of the wall inertial force will be combined with 100% of the seismic active earth loading and 100% of the wall inertial force will be combined with 50% of the seismic active earth loading but no less than the static active earth pressure force.

- Extreme II vehicle impact will be ignored for stability calculations because based on engineering judgement, the section of barrier that is hit will not detach from the rest of the barrier.

 PARSONS		MADE BY		DATE		CHK BY		DATE					
		B. Christophersen		11/23/2021		N. Ala		11/24/2021					
Job Number	WBS Number	TITLE	WSDOT I-405 - Renton to Bellevue										
650512	00515		Soil Retaining Barrier Design										

Effect	P	M	SERVICE	STRENGTH		EXTREME		
			SER 01	STR 01-01	STR 01-02	EE 01-01	EE 01-02	EE 01-03
			$\gamma$	$\gamma$ Max	$\gamma$ Min	$\gamma$	$\gamma$	$\gamma$
#	k / ft	k-ft / ft	#	#	#	#	#	#
DC	1.80	0	1.00	1.25	0.90	1.00	1.00	1.00
EV	0.43	-0.48	1.00	1.00	1.00	1.00	1.00	1.00
EH <sub>a</sub>	0	1.39	1.00	1.50	0.90	0.00	0.00	1.00
EH <sub>p</sub>	0	-1.00	0.00	0.00	0.00	1.00	1.00	1.00
EH <sub>AE</sub>	0	3.13	0.00	0.00	0.00	1.00	0.50	0.00
IR <sub>DC</sub>	0	1.10	0.00	0.00	0.00	0.50	1.00	1.00
e	-	-	4.89 in	7.18 in	4.51 in	11.88 in	6.41 in	5.45 in

e <sub>range</sub>	e <sub>min,lim</sub>	e <sub>max,lim</sub>	e <sub>max</sub>	Check
in	in	in	in	Text
24.33	-12.167	12.167	7.18	OK

$$\begin{aligned} e_{\text{range}} &= 2/3B && \{\text{two-thirds eccentricity limit}\} && [\text{AASHTO 11.6.3.3}] \\ e_{\text{min,lim}} &= -e_{\text{range}} / 2 && \{\text{minimum eccentricity limit from center of footing}\} && [\text{AASHTO 11.6.3.3}] \\ e_{\text{max,lim}} &= e_{\text{range}} / 2 && \{\text{maximum eccentricity limit from center of footing}\} && [\text{AASHTO 11.6.3.3}] \end{aligned}$$

- Since live load effects are factored by 0.50 in the EE01 load combination, the eccentricity limit is determined based on linear interpolation between 2/3 and 8/10 of the base of the wall per [AASHTO 11.6.5.1].

e <sub>lim,2/3</sub>	e <sub>lim,8/10</sub>	$\gamma_{LL,EE01}$	e <sub>range</sub>	e <sub>min,lim</sub>	e <sub>max,lim</sub>	e <sub>max</sub>	Check
in	in	#	in	in	in	in	Text
24.33	29.20	0.00	24.33	-12.17	12.17	11.88 in	OK

$$\begin{aligned} e_{\text{min}} &= B_{\text{bot}} / 2 - e_{\text{lim}} / 2 && \{\text{minimum eccentricity limit from center of wall base}\} && [\text{AASHTO 11.6.3.3 \& 11.6.5}] \\ e_{\text{max}} &= B_{\text{bot}} / 2 + e_{\text{lim}} / 2 && \{\text{maximum eccentricity limit from center of wall base}\} && [\text{AASHTO 11.6.3.3 \& 11.6.5}] \end{aligned}$$

 PARSONS		MADE BY	DATE	CHK BY	DATE
		B. Christophersen	11/23/2021	N. Ala	11/24/2021
Job Number	WBS Number	TITLE	WSDOT I-405 - Renton to Bellevue		
650512	00515		Soil Retaining Barrier Design		

2. Check footing sliding (cohesionless soil) at service and strength limit states [AASHTO 10.6.3.4].

$\theta_f =$	36.0 °	{internal soil friction angle}	
$\phi_{t,ser} =$	1.00	{service limit state sliding resistance factor}	[AASHTO 10.5.5.1]
$\phi_{t,str} =$	0.80	{strength limit state sliding resistance factor}	[AASHTO Table 10.5.5.2.2-1]
$\phi_{ep,ser} =$	1.00	{service limit state passive resistance factor}	[AASHTO 10.5.5.1]
$\phi_{ep,str} =$	0.50	{strength limit state passive resistance factor}	[AASHTO Table 10.5.5.2.2-1]
$\phi_{t,ee} =$	1.00	{extreme event limit state sliding friction resistance factor}	[AASHTO 10.5.5.3.3]
$\phi_{ep,ee} =$	1.00	{extreme event limit state sliding passive pressure resistance factor}	[AASHTO 10.5.5.3.3]

- Vertical force effects for determining sliding resistance are not factored since a resistance factor is applied to the overall sliding resistance. Vertical live loads are conservatively ignored.

Effect	P	$\gamma$
#	k / ft	#
DC	1.80	1.00
EV	0.43	1.00
EH <sub>a</sub>	0	1.00
EH <sub>p</sub>	0	1.00
EH <sub>AE</sub>	0	0.00
IR <sub>DC</sub>	0	0.00
<b>R<sub>t</sub></b>	-	<b>1.29 k/ft</b>

Limit State	R <sub>t</sub>	R <sub>ep</sub>	$\phi R$
Text	k / ft	k / ft	k / ft
SER	1.29	1.50	<b>2.79</b>
STR	1.29	1.50	<b>1.78</b>
EXT	1.29	1.50	<b>2.79</b>

$$\phi R = \{\text{factored sliding resistance}\}$$

$$R_t = |V \tan \theta_f| \quad \{\text{total wall sliding friction resistance}\} \quad [\text{AASHTO 10.6.3.4-2}]$$

Effect	F <sub>long</sub>	SERVICE	STRENGTH		EXTREME		
		SER 01	STR 01-01	STR 01-02	EE 01-01	EE 01-02	EE 01-03
		$\gamma$	$\gamma$ Max	$\gamma$ Min	$\gamma$	$\gamma$	$\gamma$
#	k / ft	#	#	#	#	#	#
DC	0	1.00	1.25	0.90	1.00	1.00	1.00
EV	0	1.00	1.00	1.00	1.00	1.00	1.00
EH <sub>a</sub>	0.72	1.00	1.50	0.90	0.00	0.00	1.00
EH <sub>p</sub>	0	0.00	0.00	0.00	1.00	1.00	1.00
EH <sub>AE</sub>	1.63	0.00	0.00	0.00	1.00	0.50	0.00
IR <sub>DC</sub>	0.45	0.00	0.00	0.00	0.50	1.00	1.00
$\gamma F$	-	<b>0.72 k/ft</b>	<b>1.08 k/ft</b>	<b>0.65 k/ft</b>	<b>1.86 k/ft</b>	<b>1.27 k/ft</b>	<b>1.17 k/ft</b>

F<sub>long</sub> = {horizontal longitudinal applied force effects}

$\gamma$  = {load factor}

[AASHTO Table 3.4.1-1, WSDOT BDM 3.5 & 4.2.6]

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Limit State	$\phi R$	$\gamma F$	D/C
Text	k / ft	k / ft	#
SER	2.79	0.72	0.26
STR	1.78	1.08	0.61
EXT	2.79	1.86	0.67

$\phi R$  = {factored sliding resistance}

$\gamma F$  = {factored sliding demand}

D/C =  $\gamma F / \phi R$  {demand-to-capacity ratio}

#### [J] CHECK BEARING STRESS

- Bearing resistance is not explicitly required to be checked at the Service limit state per [AASHTO 11.6.2], however it will be checked based on the settlement criteria of 1 in per [GEM].

Limit State	Bearing Capacity	$B_{fg}$	$q_r$	$q_u$	D/C
Text	#	ft	ksf	ksf	#
SER	11.00 KSF		11.00	1.00	0.09
STR	4.95 KSF	3.04	4.95	1.45	0.29
EXT	13.00 KSF		13.00	2.09	0.16

#### 1. Determine footing bearing pressure demands at Service/Strength/Extreme limit state [AASHTO 11.6.3].

Effect	P	SERVICE	STRENGTH		EXTREME		
		SER 01	STR 01-01	STR 01-02	EE 01-01	EE 01-02	EE 01-03
		$\gamma$	$\gamma$ Max	$\gamma$ Min	$\gamma$	$\gamma$	$\gamma$
#	k / ft	#	#	#	#	#	#
DC	1.80	1.00	1.25	0.90	1.00	1.00	1.00
EV	0.43	1.00	1.00	1.00	1.00	1.00	1.00
EH <sub>a</sub>	0	1.00	1.50	0.90	0.00	0.00	1.00
EH <sub>p</sub>	0	0.00	0.00	0.00	1.00	1.00	1.00
EH <sub>AE</sub>	0	0.00	0.00	0.00	1.00	0.50	0.00
IR <sub>DC</sub>	0	0.00	0.00	0.00	0.50	1.00	1.00
$\gamma P$	-	2.22 k/ft	2.67 k/ft	2.04 k/ft	2.22 k/ft	2.22 k/ft	2.22 k/ft
e	-	4.89 in	7.18 in	4.51 in	11.88 in	6.41 in	5.45 in
q	-	1.00 ksf	1.45 ksf	0.89 ksf	2.09 ksf	1.13 ksf	1.04 ksf

$$q = \gamma P / (B_{fg} - 2e)$$

{factored bearing pressure}

[AASHTO 11.6.3.2-1]

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## [K] WALL REINFORCEMENT DESIGN

### 1. Determine flexure and shear demands at base of wall for all limit states.

Effect	F <sub>L,wall,bot</sub>	M <sub>L,wall,bot</sub>	SERVICE		STRENGTH		EXTREME				
			SER 01	STR 01-01	STR 01-02	EE 01-01	EE 01-02	EE 01-03	EE 02 <sub>MID</sub>	EE 02 <sub>END</sub>	
			γ	γ Max	γ Min	γ	γ	γ	γ	γ	
#	k / ft	k-ft / ft	#	#	#	#	#	#	#	#	#
DC	0	0	1.00	1.25	0.90	1.00	1.00	1.00	1.00	1.00	1.00
EV	0	0	1.00	1.00	1.00	1.00	1.00	1.00	0.00	0.00	0.00
EH <sub>a</sub>	0.72	1.39	1.00	1.50	0.90	0.00	0.00	1.00	0.00	0.00	0.00
EH <sub>p</sub>	0	0	0.00	0.00	0.00	1.00	1.00	1.00	0.00	0.00	0.00
EH <sub>p, CT, MID</sub>	5.27	6.59	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00
EH <sub>p, CT, END</sub>	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00
EH <sub>AE</sub>	1.63	3.13	0.00	0.00	0.00	1.00	0.50	0.00	0.00	0.00	0.00
IR <sub>DC</sub>	0	1.10	0.00	0.00	0.00	0.50	1.00	1.00	0.00	0.00	0.00
CT <sub>MID</sub>	-5.42	-14.44	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00
CT <sub>END</sub>	-10.65	-28.41	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00
F <sub>L,bot</sub>	-	-	0.72 k/ft	1.08 k/ft	0.65 k/ft	1.86 k/ft	1.27 k/ft	1.17 k/ft	-0.1 k/ft	-10.7 k/ft	
M <sub>L,bot</sub>	-	-	1.4 k-ft/ft	2.1 k-ft/ft	1.2 k-ft/ft	3.7 k-ft/ft	2.7 k-ft/ft	2.5 k-ft/ft	-7.9 k-ft/ft	-28.4 k-ft/ft	

### 2. Document wall geometry and reinforcement properties.

- Positive flexure creates tension in the reinforcement closest to the wall face on the embankment side.

- Horizontal reinforcement should be placed inside vertical reinforcement.

c <sub>clr</sub> =	2.00 in	{main reinforcement clear cover}
c <sub>clr,tie</sub> =	1.50 in	{cross-tie clear cover}
c <sub>clr,tie, bot</sub> =	3.00 in	{cross-tie clear cover at bottom}

[AASHTO Table 5.10.1-1]

t <sub>wall, min</sub>	t <sub>wall, max</sub>	h <sub>wall</sub>	L <sub>wall</sub>	Wall Slope
ft	ft	ft	ft	Run/Rise
0.67	3.04	6.25	1.00	0.19

Reinforcement Properties						
Bar Direction	Description	ψ <sub>b</sub>	d <sub>b</sub>	A <sub>b</sub>	s <sub>b</sub>	A <sub>s</sub>
#	Text	#	in	in <sup>2</sup>	in	in <sup>2</sup> /ft
Horizontal	Horiz #5s	5	0.63	0.31	10.00 in	0.37
Stirrups	Vert #4s - Middle	4	0.50	0.20	18.00 in	0.13
Stirrups	Vert #4s - Ends	4	0.50	0.20	9.00 in	0.27

d<sub>s</sub> = {effective reinforcement centroid distance from furthest wall face}

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3. Check flexure at the Strength and Extreme Event limit states [AASHTO 5.6].

f <sub>y</sub>	f <sub>c</sub>	β <sub>1</sub>	ε <sub>cl</sub>	ε <sub>tl</sub>	ε <sub>cu</sub>
ksi	ksi	#	#	#	#
60	4.00	0.850	0.002	0.005	0.003

ε<sub>cl</sub> = {reinforcement compression-controlled strain limit}

[AASHTO 5.7.2.1]

ε<sub>tl</sub> = {reinforcement tension-controlled strain limit}

[AASHTO 5.7.2.1]

ε<sub>cu</sub> = {unconfined concrete ultimate strain limit}

[AASHTO 5.7.2.1]

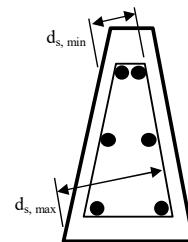
β<sub>1</sub> = for f<sub>c</sub> < 4ksi, β<sub>1</sub> = 0.85

{compression zone neutral axis ratio}

[AASHTO 5.7.2.2]

for f<sub>c</sub> > 4ksi, β<sub>1</sub> = 0.85 - 0.05\*(f<sub>c</sub> - 4) > 0.65ksi

Flexure	Location	b	A <sub>s</sub>	d <sub>s,min</sub>	d <sub>s,max</sub>
Text	Text	in	in <sup>2</sup> /ft	in	in
Positive	Middle	12.00	0.13	6.82	33.61
Negative	Middle	12.00	0.13	6.82	33.61
Negative	Ends	12.00	0.27	6.82	33.61



d<sub>s</sub> = {tensile reinforcement centroid distance from wall face}

d<sub>t</sub> = {furthest tensile reinforcement distance from wall face}

Limit State	Flexure	c	a	M <sub>n</sub>	c / d <sub>s</sub>	Check c / d <sub>s</sub> Limit	ε <sub>t</sub>	φ	φM <sub>n</sub>
#	Text	in	in	k-ft/ft	#	#	#	#	k-ft/ft
STR	Positive	0.23	0.20	22.3	0.011	OK	0.260	0.90	20.1
EXT I	Positive	0.23	0.20	22.3	0.007	OK	0.260	1.00	22.3
EXT II <sub>MID</sub>	Negative	0.23	0.20	22.3	0.007	OK	0.260	1.00	22.3
EXT II <sub>END</sub>	Negative	0.46	0.39	44.6	0.014	OK	0.128	1.00	44.6

c = (A<sub>s</sub>f<sub>y</sub>) / (0.85f<sub>c</sub>β<sub>1</sub>b) {distance from extreme compression fiber to neutral axis}

[AASHTO 5.6.3.1.1]

a = β<sub>1</sub>c

{depth of equivalent rectangular stress block}

[AASHTO 5.6.2.2]

M<sub>n</sub> = A<sub>s</sub>f<sub>y</sub> \* (d<sub>s,max</sub> - a / 2)

[AASHTO 5.7.3.2.2]

ε<sub>t</sub> = ε<sub>cu</sub> \* (d<sub>t</sub> - c) / c {net tensile strength in extreme tension steel at nominal resistance}

[strain compatibility]

φ = max (0.75 , min (0.90 , 0.75 + 0.15\*(ε<sub>t</sub> - ε<sub>cl</sub>) / (ε<sub>tl</sub> - ε<sub>cl</sub>)))

[AASHTO C5.5.4.2-1]

Limit State	Flexure	M <sub>u</sub>	D/C
#	Text	k-ft/ft	#
STR	Positive	2.08	0.10
EXT I	Positive	3.68	0.16
EXT II <sub>MID</sub>	Negative	7.85	0.35
EXT II <sub>END</sub>	Negative	28.41	0.64

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4. Check minimum wall flexural reinforcement [AASHTO 5.6.3.3].

$\lambda =$

1.00

{concrete density modification factor}

[AASHTO 5.4.2.8]

Limit State	Flexure	$f_r$	$\gamma_1$	$\gamma_2$	$\gamma_3$	$f_{cpe}$	$S_c$	$M_{cr}$	$M_{min}$	$\phi M_n$	D/C
#	Text	ksi	#	#	#	ksi	in <sup>3</sup>	k-ft/ft	k-ft/ft	k-ft/ft	#
STR	Positive	0.480	1.60	1.00	0.75	0.00	990	48	2.8	20.1	0.14
EXT I	Positive	0.480	1.60	1.00	0.75	0.00	990	48	4.9	22.3	0.22
EXT II <sub>MID</sub>	Negative	0.480	1.60	1.00	0.75	0.00	990	48	10.4	22.3	0.47
EXT II <sub>END</sub>	Negative	0.480	1.60	1.00	0.75	0.00	990	48	37.8	44.6	0.85

$$f_r = 0.24\lambda\sqrt{f'_c} \quad \{concrete modulus of rupture\}$$

[AASHTO 5.4.2.6]

$$\gamma_1 = \{\text{flexural cracking variability factor}\}$$

[AASHTO 5.6.3.3]

$$\gamma_2 = \{\text{prestress variability factor}\}$$

[AASHTO 5.6.3.3]

$$\gamma_3 = \{\text{ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement}\}$$

[AASHTO 5.6.3.3]

$$f_{cpe} = \{\text{compressive stress in concrete due to effective prestress force only}\}$$

[AASHTO 5.6.3.3]

$$S_c = b t_{\text{wall}}^2 / 6 \quad \{\text{section modulus per unit length of wall}\}$$

[AASHTO 5.6.3.3]

**5.6.3.3—Minimum Reinforcement**

Unless otherwise specified, at any section of a noncompression-controlled flexural component, the amount of prestressed and nonprestressed tensile reinforcement shall be adequate to develop a factored flexural resistance,  $M_r$ , greater than or equal to the lesser of the following:

- 1.33 times the factored moment required by the applicable strength load combination specified in Table 3.4.1-1;

$$\bullet \quad M_{cr} = \gamma_3 \left[ (\gamma_1 f_r + \gamma_2 f_{cpe}) S_c - M_{min} \left( \frac{S_c}{S_{uc}} - 1 \right) \right]$$

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**5. Check wall crack control at the Service limit state [AASHTO 5.6.7].**

n =	6.8	= $E_s / E_c$
$\gamma_e =$	1.00	{class 1 exposure condition} [AASHTO 5.6.7]

A <sub>s</sub>	b	d <sub>s</sub>	d <sub>c</sub>	$\rho$	k	j	d <sub>NA</sub>
in <sup>2</sup> /ft	in	in	in	#	#	#	in
0.13	12.00	20.22	1.75	0.0005	0.083	0.972	1.67

$d_c = d - d_s$  {thickness of concrete cover measured from extreme tension fiber to center of flexural reinforcement}  
 $\rho = A_s / (bd_s)$  {ratio of effective area of tension reinforcement to effective area of concrete}  
 $k = \sqrt{(2pn + (pn)^2) - pn}$  {ratio of depth of neutral axis to effective depth, d<sub>s</sub>}  
 $j = 1 - k / 3$  {ratio of lever arm of resisting couple to depth, d<sub>s</sub>}  
 $d_{NA} = kd_s$  {depth of netrual axis from extreme compression surface}

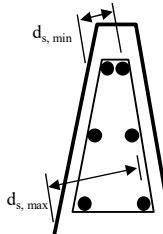
M <sub>u,SER</sub>	f <sub>ss</sub>	$\beta_s$	s <sub>max</sub>	s <sub>prov</sub>	D/C
k-ft/ft	ksi	#	in	in	#
1.39	6.35	1.12	18.00	18.00	1.00

$M_{u,SER} =$  {maximum Service flexural demand per unit length of wall}  
 $f_{ss} = M_{u,SER} / (A_s j d_s)$   
 $\beta_s = 1 + d_c / (0.7 * (d - d_c))$  {ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face} [AASHTO 5.6.7-2]  
 $s_{max} = (700\gamma_e) / (\beta_s f_{ss}) - 2d_c$  [AASHTO 5.6.7-1]

**6. Check moment capacity to be used for critical wall length in the vehicle impact calculation.**

b	# of Bars	A <sub>s</sub>	d <sub>s,min</sub>	d <sub>s,max</sub>
in	#	in <sup>2</sup>	in	in
76.34	8.00	2.98	6.26	33.05

$d_s =$  {tensile reinforcement centroid distance from wall face}



c	a	M <sub>n</sub>	c / d <sub>s</sub>	Check c / d <sub>s</sub> Limit	$\varepsilon_t$	$\varphi$	$\varphi M_n$
in	in	k-ft	#	#	#	#	k-ft
0.81	0.69	287.3	0.041	OK	0.070	0.90	258.6

$c = (A_s f_y) / (0.85 f_c \beta_1 b)$  {distance from extreme compression fiber to neutral axis}  
 $a = \beta_1 c$  {depth of equivalent rectangular stress block}  
 $M_n = A_s f_y * (d_{s,max} - a / 2)$   
 $\varepsilon_t = \varepsilon_{cu} * (d_t - c) / c$  {net tensile strength in extreme tension steel at nominal resistance}  
 $\phi = \max (0.75, \min (0.90, 0.75 + 0.15 * (\varepsilon_t - \varepsilon_{el}) / (\varepsilon_{el} - \varepsilon_{cl})))$  [strain compatibility] [AASHTO C5.5.4.2-1]

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7. Check shear design at the Strength, Service, and Extreme limit state [AASHTO 5.7].

F <sub>u,STR</sub>	F <sub>u,SER</sub>	F <sub>u,EXT, I</sub>	F <sub>u,EXT, II, MID</sub>	F <sub>u,EXT, II, END</sub>
k/ft	k/ft	k/ft	k/ft	k/ft
1.08	0.72	1.86	0.14	10.65

b <sub>v</sub>	h	d <sub>s</sub>	d <sub>v</sub>
in	in	in	in
12.00	22.25	20.22	18.19

b<sub>v</sub> = {effective width of section measured parallel to neutral axis}

h = {thickness of the section in the direction of loading}

d<sub>s</sub> = {depth to the center of flexural reinforcement in the direction of loading}

d<sub>v</sub> = MAX( 0.72\*h, 0.9\*d<sub>s</sub>) [AASHTO 5.7.2.8]

ϕ<sub>s</sub> = 0.9 [AASHTO 5.5.4.2]

Limit State	Flexure	ε	β	θ	V <sub>c</sub>	Is Transverse Reinforcement Required?
#	Text	#	#	Degrees	k/ft	
STR	Positive	-	2	45	27.60	Transverse Reinforcement Not Required
SER	Positive	-	2	45	27.60	Transverse Reinforcement Not Required
EXT I	Positive	-	2	45	27.60	Transverse Reinforcement Not Required
EXT II <sub>MID</sub>	Negative	-	2	45	27.60	Transverse Reinforcement Not Required
EXT II <sub>END</sub>	Negative	-	2	45	27.60	Transverse Reinforcement Not Required

ε<sub>s</sub> = {net longitudinal tensile strain at the centroid of tension reinforcement} [AASHTO 5.7.3.4.2-4]

β = {factor indicating ability of diagonally cracked concrete to transmit tension and shear} [AASHTO 5.7.3.4.1]

θ = {angle of inclination of diagonal compressive stresses} [AASHTO 5.7.3.4.1]

V<sub>c</sub> = 0.0316 \* β \* √f<sub>c</sub> \* b<sub>v</sub> \* d<sub>v</sub> {shear resistance of concrete} [AAHSTO 5.7.3.3-3]

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Limit State	Flexure	V <sub>u</sub>	S <sub>max</sub>	D/C	A <sub>v</sub> , min	D/C
#	Text	ksi	in	#	in <sup>2</sup> /ft	#
STR	Positive	0.006	14.55	N/A	0.23	N/A
SER	Positive	0.004	14.55	N/A	0.23	N/A
EXT I	Positive	0.009	14.55	N/A	0.23	N/A
EXT II <sub>MID</sub>	Negative	0.001	14.55	N/A	0.23	N/A
EXT II <sub>END</sub>	Negative	0.001	14.55	N/A	0.23	N/A

$$V_u = V_u / (\phi * b_v * d_v)$$

{shear stress in concrete}

[AASHTO 5.7.2.8-1]

$$S_{max} = IF[ V_u < 0.125 * f_c, MIN( 0.8 * d_v, 24in ), MIN( 0.4 * d_v, 12in ) ]$$

{max spacing of shear reinforcement}

[AASHTO 5.7.2.6-1/2]

$$A_{v,min} = 0.0316 * \sqrt{f_c} * b_v * s / f_y$$

{minimum area of transverse reinforcement}

[AASHTO 5.7.2.5-1]

Limit State	Flexure	V <sub>s</sub>	V <sub>n</sub>	φV <sub>n</sub>	D/C
#	Text	k/ft	k/ft	k/ft	#
STR	Positive	10.87	38.47	34.62	0.03
SER	Positive	10.87	38.47	34.62	0.02
EXT I	Positive	10.13	37.73	33.95	0.05
EXT II <sub>MID</sub>	Negative	7.87	35.46	31.92	0.00
EXT II <sub>END</sub>	Negative	8.79	36.38	32.75	0.33

$$V_s = A_v * f_y * d_v * \cot(\theta) / s$$

{shear resistance of transverse reinforcement}

[AASHTO 5.7.3.3-4]

$$V_n = MIN( 0.25 * f_c * b_v * d_v, V_c + V_s )$$

{nominal shear resistance}

[AASHTO 5.7.3.3-1/2]